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Environmental Planning Division
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Athabasca River Power Development

Crooked Rapids Dam

Feasibility Study

Volume 1

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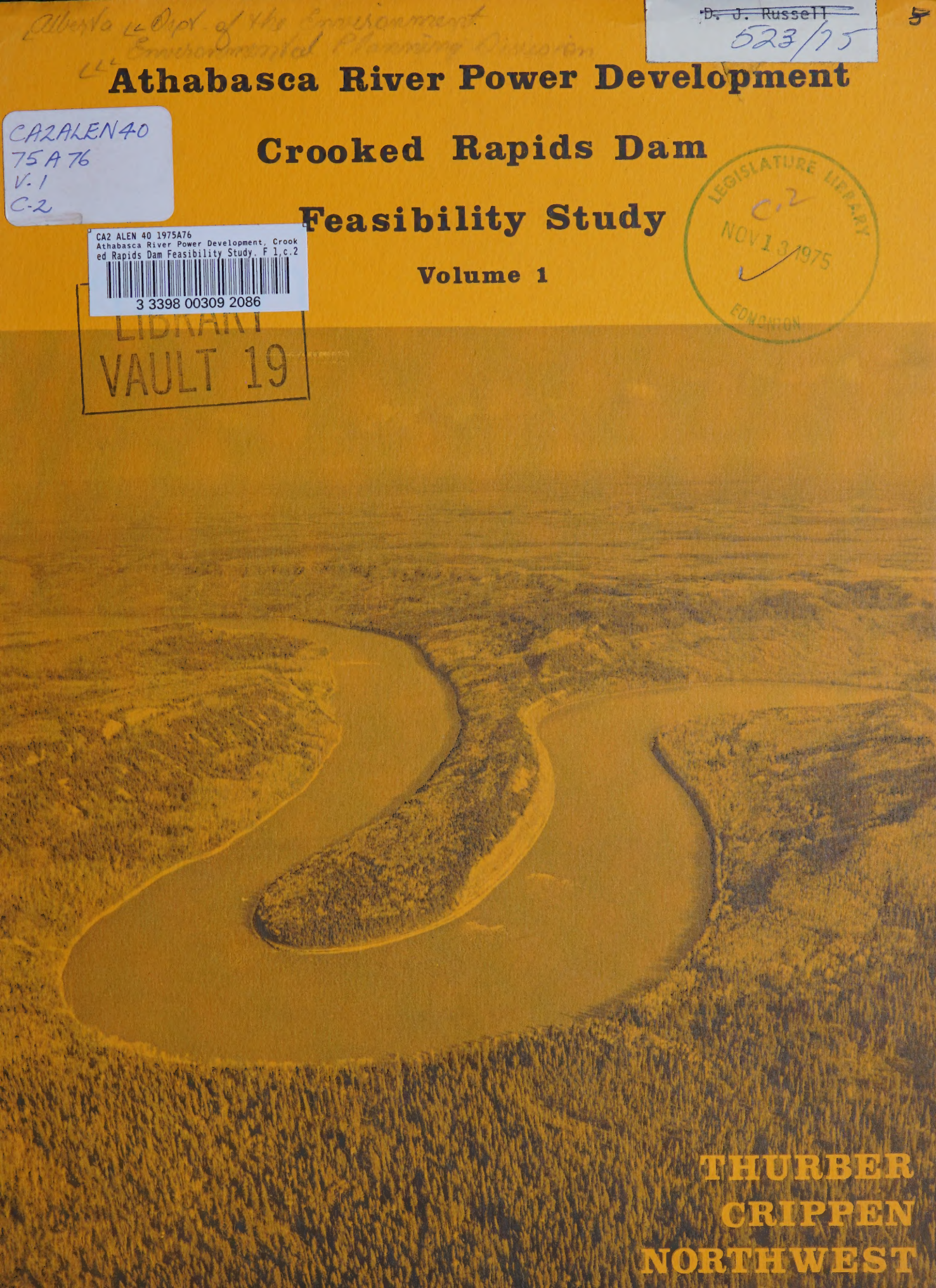

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ATHABASCA RIVER POWER DEVELOPMENT
CROOKED RAPIDS DAM
FEASIBILITY STUDY

Report to

ALBERTA DEPARTMENT OF THE ENVIRONMENT
ENVIRONMENTAL PLANNING DIVISION

Volume 1
Text

Thurber Consultants Ltd.
Crippen Engineering Ltd.
Northwest Hydraulic Consultants Ltd.

February, 1975

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File: 15-14-1

Mr. Ben Doell, P. Eng.,
Environmental Basin Planner,
Environmental Planning Division,
Department of the Environment,
Province of Alberta,
10405 - 100th Avenue,
EDMONTON, Alberta

Dear Mr. Doell:

Athabasca River Power Development
Crooked Rapids Site Study

We are pleased to submit herewith our report "Athabasca River Power Development, Crooked Rapids Dam Feasibility Study" in accordance with our agreement dated March 6th, 1974 and amendments dated January 22nd, 1975 and March 4th, 1975.

We wish to express our appreciation for the contributions made throughout the study by yourself and by other members of the Environmental Planning Division. We should also like to acknowledge the help given by Mr. C. Goodman, P. Eng. and Mr. J. Yu, P. Eng. of the Energy Resources Conservation Board.

Yours very truly,

THURBER CONSULTANTS LTD.

Sydney F. Hillis

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Abstract

This report presents the results of a preliminary study carried out to determine the technical feasibility and estimated cost of constructing a dam on the Athabasca River at the location known as Crooked Rapids, about 26 miles upstream from Fort McMurray. The dam was to be considered primarily as a potential source of hydro-electric power, but its usefulness as a flood control and water supply structure was also to be taken into consideration.

The study was to assume that the dam would be the first development on the river, and, as a result, it would not have the benefit of upstream storage.

The study encompassed all engineering and economic aspects of alternate dam alignments and heights in the general vicinity of Crooked Rapids. It included such items as site mapping, hydrological, geological and borrow area investigations, shoreline stability studies, river ice and reservoir sedimentation studies, power potential studies, and capital and energy cost estimates. A limited assessment of the upstream and downstream effects of constructing a dam at Crooked Rapids, for example the effect on flooding at Fort McMurray, was also made. Detailed environmental implications were not considered since it was felt that it would be premature to study these until it had been established if, in fact, it was technically feasible to construct a dam at the postulated site.

On the basis of topographical features, two locations in the vicinity of Crooked Rapids were identified as being most favourable for consideration as damsites. One of the locations is suitable for a Medium-Head Dam, up to about 300 ft in height, while the other is suitable for a High-Head Dam about 550 ft in height. After consideration of various factors, including a desire to have a substantial difference in height between the

two dams, it was decided to study projects incorporating a 255-ft high dam at the first location, and a 540-ft high dam at the second.

The studies carried out show that although the two sites are topographically suitable for development there are severe technical problems associated with both, and in fact that problems exist throughout the entire Crooked Rapids area. Most important amongst the problems are those related to the abutment foundation conditions, which are extremely poor.

The poor abutment conditions, caused largely by the presence of substantial thicknesses of uncemented fine sands and silts, and even a thick layer of highly plastic clay and clay-shale at the High-Head Project site, rule out the use of a concrete dam of any type. Any dam constructed in the vicinity of Crooked Rapids would thus have to be of earth or a combination of earth and rock. However, the studies also revealed that there are no sizeable deposits of good earth borrow materials near Crooked Rapids. Hence, a dam would have to be constructed largely from locally excavated rocks which would include the Athabasca Tar Sands, sandy materials from required project excavations, and quarried limestone. The limestone, although by far the most competent rock in the area, is fragmental and shaley, factors which detract from its suitability as fill material.

The severe technical problems associated with a hydro-electric project at Crooked Rapids resulted in project layouts being developed which include several features which are unprecedented. These include the 50-ft excavated diameter diversion/power tunnels through the Tar Sands, the use of the Tar Sands as an impervious core of the dam, and the extremely deep and extensive seepage cut-off wall for the High-Head Project.

The known and potential problems at both postulated sites are of such a magnitude that a much more detailed investigation, especially for the

High-Head Project, would be required before it could definitely be determined if it is feasible to develop either. Such investigation would include exploratory drilling, exploratory adits, laboratory testing and further design studies.

The above notwithstanding, it is believed with a reasonable degree of confidence that the problems associated with the Medium-Head Project could be overcome and the project could be constructed within the capital cost estimated.

However, the same cannot be said for the High-Head Project where many of the problems are considerably more severe than for the Medium-Head Project. Because of the more severe problems and the resulting greater uncertainties, the High-Head Project layout developed must at this stage of the investigation be considered "conceptual" only.

The total capital cost of the postulated projects in terms of January, 1975 dollars is estimated at about 400 million for the Medium-Head Project and 1.1 billion for the High-Head Project.

For the Medium-Head Project, the estimated cost per installed kW is $3,320, and the estimated firm and total energy costs, not including transmission, are 76 mills and 46 mills per kWh, respectively. These costs are extremely high.

For the High-Head Project, the estimated cost per installed kW is $2,000, and the estimated firm and total energy costs, not including transmission, are 30 mills and 24 mills per kWh, respectively. These costs although much less than for the Medium-Head Project are still very high when compared with current costs.

Although limited studies indicate that there would be some non power-related

benefits from construction of a dam at Crooked Rapids, for example the potential for flooding at Fort McMurray would be decreased, and navigation channels would tend to be improved, these do not appear to be substantial enough to be significant factors in justifying the very high capital expenditures which would be involved. Such expenditures would have to be justified mainly on the basis of benefits derived from the energy output. Although more would have to be known regarding the specific uses for the power, it is unlikely, considering the energy costs given above, that a development at Crooked Rapids, without the benefit of upstream storage, could be justified at this time.

It is recommended, therefore, that before detailed studies are carried out at Crooked Rapids, an overall appraisal of the Athabasca River be made in an effort to locate alternate sites to Crooked Rapids where geological conditions and the availability of borrow materials are more favourable, and/or to locate sites which could provide a development at Crooked Rapids with upstream storage, thus making development there more economical.

Such an appraisal would comprise an airphoto study of the entire river, geological mapping at selected locations, and preliminary feasibility studies at probably two or three potential damsites.

1. INTRODUCTION

1.1 Study Authorization

In March 1974, the Department of the Environment of the Province of Alberta retained Thurber Consultants Ltd., who were to collaborate with Crippen Engineering Ltd. and Northwest Hydraulic Consultants Ltd., to undertake a study to determine the feasibility of constructing a dam on the Athabasca River at the location known as Crooked Rapids. This report presents the results of the study. The study was to be for the joint use of the Environmental Planning Division of the Alberta Department of the Environment and the Alberta Energy Resources Conservation Board.

The report is in two volumes, the basic information being given in Volume 1, while the detailed data are presented in Volume 2. Volume 1 of the report has been prepared such that, if desired, the study can be understood and the conclusions appreciated without reference to Volume 2.

1.2 Study Area

The Athabasca River is one of the larger rivers in Alberta. It rises in the Rocky Mountains near Mount Columbia and flows for nearly 1,000 miles in a generally northeasterly direction, whence it empties into Lake Athabasca. Thereafter its waters are conveyed by the Slave River to Great Slave Lake, and from there to the Arctic Ocean by way of the McKenzie River.

About 140 miles upstream from where the river empties into

Lake Athabasca, it passes through the developing town of Fort McMurray. As will be noted from Fig. 1, the Crooked Rapids Damsite Study Area is located about 26 miles upstream from Fort McMurray.

Physiographically the Damsite Study Area lies within the Alberta Plateau of the Interior Plains. In the vicinity of Crooked Rapids the elevation of the plateau is some 1,500 ft above sea level. The Athabasca River in the Damsite Study Area is incised into the plateau to an elevation of about 900 ft above sea level. Small lakes and meandering streams drain the plateau which is covered by black spruce and muskeg. The valley of the Athabasca River itself is vegetated by spruce, poplar and birch. The only present access to the Damsite Study Area is by helicopter although a winter trail passes quite close by. A general view of the Area is shown on Photo 1.

1.3 Previous Studies

The Athabasca River has been examined from time to time as a potential source of hydro-electric power, and reports dating back to 1911 describing the various investigations are available. However, discussion of these and their implications are outside the scope of this report. It is sufficient to say here that the most detailed studies, although still very preliminary, were carried out in the early 1950s by Calgary Power Ltd. working in conjunction with the Alberta Power Commission, and that, amongst other things, these studies concluded that one of the most promising damsites was at Crooked Rapids.

2. TERMS OF REFERENCE

In essence, the study was to be concerned only with the technical feasibility of constructing a dam at Crooked Rapids. Detailed environmental implications were not to be considered since it was not at all certain that it was technically feasible to construct a dam at this location, and as a result, it was felt premature to be concerned about the environmental consequences until this point had been reasonably established.

As the title of the report implies, the dam was to be considered primarily as a potential source of hydro-electric power, but its usefulness as a flood control and water supply structure was also to be taken into consideration.

Owing to budgetary and time restrictions, and bearing in mind the poor accessibility to and complexity of the potential dam site, it was understood that the feasibility study would have to be "preliminary" rather than "detailed".

A most important point in the terms of reference was that the study was to assume that any dam which might be constructed at Crooked Rapids would be the first development on the river. That is to say, it was to be assumed that immediately after construction, the project would not have the benefit of any upstream storage.

In previous studies of the Crooked Rapids site it had always been assumed that the dam would be located near the downstream end of the Crooked Rapids meander loop, near what is shown on Fig. 2 as the Medium-Head Site, so that advantage could be taken of the favourable topography in constructing the diversion works, the spillway and the powerhouse. In fact it would appear that this

reason combined with the known presence of a relatively competent limestone in the riverbed downstream of Crooked Rapids was why previous studies indicated that the Crooked Rapids Site was probably the most promising on the river. However, it is apparent from the topography shown on Fig. 2 that a site near the downstream end of the Crooked Rapids meander loop is only suitable for a dam up to 250 to 300 ft in height. Otherwise the topographic advantages mentioned above disappear and the dam crest becomes excessively long. Some 5 miles upstream, however, considering topography only, a dam about twice this height could be constructed.

Very early in the present study it became clear that, since no upstream storage was to be assumed, a 500 to 600 ft high dam would yield much less expensive power than a 250 to 300 ft high dam, providing that the unit costs of construction and the problems did not increase greatly with increase in height of the dam. At the same time, however, it also became clear that there were major technical problems at Crooked Rapids and that these increased significantly as the height of the dam was increased. As a result, it was decided to study dams of two significantly different heights, which as discussed above meant that two different dam axes had to be considered. These axes are shown on Fig. 2 as the High-Head Site and the Medium-Head Site, terminology which will be used throughout the report.

Views of the two sites are shown on Photos 1, 2 and 3.

3. HYDROLOGY

3.1 General

The results of the hydrologic studies for the potential damsites are given in detail in Appendix A.

The studies were separated into three main areas as follows:-

3.2 Derivation and Compilation of Set of Historical Mean Monthly Flows

The results of this study are given in Table 1.

This table shows the mean monthly flows covering the 60-year period from 1914 through 1973. Since there is no gauging station at Crooked Rapids and since the records at the gauging stations which had to be used (principally the stations at the towns of Athabasca and Fort McMurray) are not continuous throughout the 60-year period, the flows given in Table 1 had to be estimated using standard methods of drainage area adjustment and correlation.

The following may be noted from Table 1.

i The annual mean monthly flow is 17,000 cfs.

ii The maximum monthly flow is 89,000 cfs and this occurred in May 1948.

iii The minimum monthly flow is 1980 cfs and this occurred in February 1964.

Table 1

Estimated Monthly Flows in cfs

Athabasca River at Crooked Rapids

YEAR	JANUARY	FEBRUARY	MARCH	APRIL	MAY	JUNE	JULY	AUGUST	SEPTEMBER	OCTOBER	NOVEMBER	DECEMBER	ANNUAL MEAN
1914	3890	3420	3640	6050	18600	56600	47800	23000	17100	16400	7450	3580	17300
1915	4340	3730	4420	12500	18500	43700	63000	31900	18300	12500	7060	3720	18600
1916	3350	3070	3150	8750	19200	32500	51200	26300	18800	13800	7410	3350	15900
1917	3520	2820	2710	8860	48500	43000	30700	21000	15000	14900	7000	3750	17400
1918	4500	3630	3590	11300	19800	40700	41400	22100	15000	10900	5980	3330	15200
1919	3200	2890	2560	7970	14300	28900	29900	23900	22100	11800	6830	3890	13200
1920	3730	3420	3310	5910	54200	46200	49100	32500	21200	17800	7790	3940	20800
1921	4630	4590	4680	13100	34100	41000	37300	27500	17900	14300	9580	4030	17700
1922	3360	3260	3350	6400	24900	28300	28600	23300	17500	12700	6480	2580	13400
1923	2680	2300	2360	7810	21000	39600	43000	28000	20600	15900	10100	5170	16500
1924	3040	3610	3370	5230	34300	29800	34300	32500	28700	18100	9770	3570	17200
1925	2590	2520	3470	28400	30200	37100	32300	33200	27400	18700	9350	6090	19300
1926	5970	4910	5030	15500	22100	30100	31800	20300	26100	19600	11000	5180	16500
1927	4350	4130	4500	11700	32800	43400	55400	28300	23100	17700	8560	4450	19900
1928	4640	3890	4680	11400	31300	47100	51000	26300	17800	13800	7370	5260	18700
1929	3810	3310	3990	8980	28800	33200	30200	22900	16700	16000	8250	3360	15000
1930	2830	2920	3190	11800	28400	45100	41500	25600	19400	12500	9130	5740	17300
1931	4170	3910	4560	8110	23700	34800	42600	25600	19500	14400	7690	4820	14000
1932	4010	3380	3100	11800	30600	49400	37000	27300	19900	14300	7530	4210	17700
1933	4150	3550	3080	11300	36100	37800	38300	25700	19900	12500	6490	3650	16900
1934	3950	3580	3350	11900	29100	35300	32200	22900	16200	13300	7100	4200	15300
1935	3050	3170	4720	11900	28900	40000	50200	29300	19000	13800	9360	3900	18100
1936	3720	3460	3050	17100	34600	38500	33200	24800	17800	13100	7450	3370	16700
1937	2650	2490	2940	7910	25200	33300	40100	24100	18800	14200	8880	4520	15400
1938	3860	2850	3270	8460	18600	33500	32900	24200	21100	10100	5330	3260	14000
1939	3050	2880	2990	9650	20400	23600	33500	20200	11500	11800	6200	3620	12500
1940	3330	3100	3210	14800	39200	35000	35600	21800	17300	12400	6510	3750	16300
1941	3430	3170	3290	8160	17600	31200	36500	23500	17600	15800	8230	4400	14400
1942	4020	3620	3740	9220	17200	43200	43900	28000	24100	14100	7300	4120	16000
1943	3730	3400	3510	16600	21100	42800	45600	27100	15800	10900	5760	3430	16600
1944	3190	2990	3100	9400	26300	74400	46300	27800	19900	14400	7550	4190	20000
1945	3780	3440	3560	7800	20000	27200	25000	21300	12800	13500	7090	4000	12500
1946	3630	3320	3440	10900	19200	42400	34900	21200	14400	8800	4690	3000	14200
1947	2850	2730	2840	13800	31300	36300	35300	29900	22100	16300	8490	4600	17200
1948	4110	3690	3800	13200	89000	46800	41200	38100	25100	16500	8590	4650	24100
1949	4140	3720	3830	10400	17900	24700	28500	26800	15500	9700	5140	3180	12800
1950	2990	2840	2950	9510	26100	36000	39200	23400	15300	9000	4760	3030	14600
1951	2870	2740	2860	9510	43600	29200	41300	25900	16400	11300	6970	3360	15000
1952	3340	3160	3080	17700	20700	44400	44200	24700	16400	12500	6700	2760	16600
1953	2800	2980	3370	8300	33400	43800	39700	37000	27300	13700	7900	3570	18700
1954	3790	3670	3630	5200	49600	75100	50700	44700	46700	21500	11200	6790	26900
1955	3880	3260	3250	12600	29400	48000	45400	25800	15900	11400	4920	3760	17300
1956	4160	3620	3370	18300	30100	44200	33600	26300	17100	12300	5310	3300	16800
1957	2670	2890	3690	9800	33700	28800	30700	28700	20600	16700	12200	6030	16400
1958	5470	4450	4510	23900	30700	32100	34700	17300	13300	9350	4930	2830	15300
1959	2440	2170	2580	8570	15600	35600	34500	18600	19900	12900	5410	4650	13600
1960	3200	3010	2870	10700	24100	40100	53400	23900	22800	14900	10600	4740	17900
1961	4290	3570	3800	9050	18300	32700	34800	23000	12300	11300	7000	3670	13700
1962	3490	2960	2690	16200	43800	44800	41400	29100	20000	12300	8760	5200	19200
1963	4280	4430	4460	21100	40300	36400	28900	20900	17100	12700	7380	2360	16700
1964	2200	1980	2620	4460	24000	33500	32700	30200	24900	25900	9740	4040	16400
1965	5240	4440	4710	11100	47500	53200	72400	34400	34400	24000	10900	6670	23900
1966	5250	5490	5570	9000	32600	41700	38400	34600	29600	15100	9280	6160	19400
1967	4940	4130	3300	8560	34200	42600	31800	24900	15400	9920	5930	2480	15700
1968	2760	2960	3960	6770	14300	28900	33600	27300	18000	11600	6430	3340	13300
1969	2790	2290	2020	4560	26400	28800	28700	37200	24900	20600	9000	5070	16000
1970	3310	3590	3820	14600	23700	34500	50300	24900	17100	12500	5500	3330	16400
1971	3030	2990	2960	16300	29100	55500	85600	33600	19300	16400	7850	4380	23100
1972	5220	4600	5120	14800	43100	59200	46700	29000	18700	17000	8740	4340	21400
1973	4850	4120	4110	10800	32800	55400	38900	29900	18300	18300	9960	6590	19500
Mean	3710	3320	4110	12600	29500	40000	41100	26900	19900	14300	7710	4150	17000

iv. Generally, the maximum monthly flow occurs in June or July and the long term mean value is about 40,000 cfs.

v. Generally, the minimum monthly flow occurs in January or February and the long term mean value is about 3500 cfs.

vi. The wettest year was 1954 and the driest 1939, when the annual mean monthly flows were 26,900 cfs and 12,500 cfs, respectively.

3.3 Flood Frequency Analysis

The results of the flood frequency analyses are shown on Fig. 3 The hydrographs shown on this figure were obtained from a statistical appraisal of the maximum annual flood hydrographs, from the gauge records at the town of Athabasca adjusted for the difference in drainage area between Athabasca and Crooked Rapids. Flood peaks having return periods of 2, 25, 50 and 100 years were estimated to be 80,000, 172,000, 196,000 and 219,000 cfs, respectively. The total discharge for a 12-day period during floods having return periods of 2, 25, 50 and 100 years, were estimated to be 1.31, 2.28, 2.56 and 2.82 million ac-ft respectively.

3.4 Maximum Probable Flood

The estimated Maximum Probable Flood hydrograph is also shown on Fig. 3.

In developing this hydrograph, it was assumed that the

critical hydrometeorological factors which would generate a Maximum Probable Flood would be the occurrence in late July or early August of a Maximum Probable Storm centered over the Athabasca River basin upstream of the Pembina River confluence, combined with a second large storm (not necessarily as great as the Maximum Probable Storm for the area) between the Pembina confluence and Crooked Rapids occurring a few days after the Maximum Probable Storm above the Pembina Confluence. Because of the time of the year, it was assumed that prior to storm development the groundwater and depression storage capacities would be relatively high and river base flows relatively low. On the other hand, the storms would be extremely severe since the mid-summer air masses would be capable of storing a large amount of precipitable water. The Maximum Probable Storm was taken as the least upper bound on the realistic upper limit of precipitation from a nearby large historical storm, after transposing and adjusting for maximum moisture content. The run-off coefficients used and adjustment of the hydrograph to account for differences in drainage areas between the storm centres and Crooked Rapids were in accordance with normal hydrologic practice.

It will be noted from Fig. 3 that the peak of the flood occurs about 5 days after direct run-off in the headwaters, that the peak discharge is 570,000 cfs, and that the volume of the 12-day flood is 2.935 million cfs-days, or 5.87 million ac-ft.

Two checks were made on the value of peak discharge shown on Fig. 3. First, a comparison was made with the empirical curve for prediction of mean daily peak discharge developed by the Saskatchewan - Nelson Basin Board, and secondly a statistical

analysis was made of recorded peak flows at the town of Athabasca.

In comparison with the Saskatchewan - Nelson Basin Board curve the peak discharge of 570,000 cfs is slightly conservative in that this discharge is equivalent to 14.8 cfs/sq mi of drainage area compared with the 13 cfs/sq mi obtained from the curve. The statistical analysis of historical peaks at the town of Athabasca indicates that on a normal distribution basis, a flow of 570,000 cfs at Crooked Rapids is about the upper 90 per cent confidence limit for the 10,000-year flood. Again this would indicate that the peak discharge shown on Fig. 3 is somewhat conservative. However, it is not unduly conservative, and since it is based on an up-to-date rational approach, 570,000 cfs has been accepted as the Maximum Probable Flood for the purposes of this study.

4. DAMSITE GEOLOGY

4.1 General

The results of the geological investigation in the Damsite Study Area are given in detail in Appendix B, and a summary is presented on Fig. 4.

It should be noted that no exploratory drilling was carried out during the damsite geology study. The information obtained was based on existing data, on an airphoto study, and on field mapping of four bedrock exposures in the Damsite Study Area Geological Sections S1 through S4 shown on Fig. 4.

In the Damsite Study Area from about the riverbed level to a depth of some 1700 ft the geological section comprises Devonian carbonates, evaporites and shales. Above the Devonian surface lie 500 to 600 ft of almost horizontally bedded Cretaceous sandstones and shales, which in turn are overlain by possibly as much as 50 ft of Pleistocene and Recent overburden.

The Athabasca River upstream from Fort McMurray is post glacial in origin and in the Damsite Study Area it has incised itself nearly 600 ft into the Recent and Pleistocene overburden and the Cretaceous sediments. It has not cut into the Devonian members to more than about 25 ft.

During downcutting, the river formed many large meanders at various elevations in the Cretaceous sediments and many of these old meanders are obvious on airphotos and in the field. A very prominent ancient meander can be seen on Photo 1.

4.2 Stratigraphy

The stratigraphy of the sedimentary column is summarized on Fig. 4, which also shows, in general terms, the geographic distribution of the various rock formations. The lithology of the formations, working from the bottom to the top of the column is discussed below.

a. Devonian Waterways Formation

The Devonian bedrock exposed in the Damsite Study Area is the Moberly Member of the Waterways formation. It consists of a rubbly and fragmented limestone with thin interbeds of limestone and shale. Although it is heavily jointed in places, the formation is relatively competent and resistant to erosion. It is by far the most competent of all the formations exposed. In the easterly part of the Damsite Study Area about 20 ft of the Moberley Member can be seen above river level. About 1½ miles upstream from Crooked Rapids the limestone surface dips below the river, and since no drilling was carried out, the elevation of the limestone surface upstream of this location is not known. However it is believed unlikely that the Devonian limestone will be at any great depth below the riverbed anywhere within the Damsite Study Area.

b. Cretaceous McMurray Formation

This formation, which rests on the Devonian limestone, is the well-known Athabasca Tar Sands. The total thickness of the formation in the Damsite Study Area is about 120 ft. Most of the McMurray Formation in the Area is an oil-cemented quartz

sand, containing some interbeds of finer grained material and in places some interbeds of a very hard ironstone. However, about the upper 20 ft of the formation contains little or no oil.

The competence of the McMurray formation increases with oil content, a feature which increases with depth. The best of the formation can be considered a soft rock whereas the worst can better be described as a dense sand.

c. Cretaceous Clearwater Formation

This formation, which rests on the Tar Sands, is some 300 ft thick in the Damsite Study Area. It comprises interbedded argillaceous sandstones and arenaceous shales. The thickness of the interbeds vary with lithology and in general the sandstone and sand-rich beds are thicker.

Except for a few thin interbeds of competent iron-rich sandstone, ironstone, and calcareous concretionary beds, the interbedded sandstones and shales of the formation are poorly compacted and highly fractured and jointed. The sandstone is friable, generally more like a fine to medium grained dense sand or a silt than a sandstone, and the shale is fissile.

The uppermost member of the formation, estimated to be about 40 ft thick, consists of a highly plastic clay interbedded with a soft, highly fractured, highly plastic, clay shale.

d. Cretaceous Grand Rapids Formation

This formation, which rests on the Clearwater Formation, is

some 140 ft thick in the Damsite Study Area. It consists of a fine to medium grained sand to sandstone.

Although there is some interbedding of silt, clay, denser sandstone and ironstone, for the most part the material is clean, uniform, uncemented, dense sand.

e. Pleistocene and Recent Overburden

At most locations the stratigraphic column is topped by deposits of Pleistocene and/or Recent overburden. These deposits probably vary from a few feet thick to over 50 ft thick.

The Pleistocene deposits are undifferentiated glacial sand, clay and till, while the Recent deposits are undifferentiated alluvial sand and occasional gravel.

5. VALLEY SLOPE STABILITY

5.1 General

There is conspicuous evidence of slope instability all along the Athabasca River Valley upstream from Fort McMurray, and the results of a brief study of the situation between Fort McMurray and Brule Point (about 65 miles upstream) are given in Appendix C.

The study comprised a review of existing data, an airphoto evaluation and a 2-day helicopter-assisted reconnaissance, which included ground examination at 6 locations.

It is estimated that perhaps as much as 20 per cent of the area of the valley slopes is involved in active sliding at the present time, and that an additional 30 per cent has been involved previously. However, all the existing slides are shallow relative to the height of the slope, none of them extending from river level to the top of the slope. In fact no slide appears to involve the entire thickness of any one geological formation, with the probable exception of the Joli Fou Formation, discussed later. This suggests that generally the various formations in their unweathered state are not particularly unstable, or at least they contain strong horizons which inhibit the development of deep-seated slumps. The shallow slides appear to develop as the formations soften and break down as a result of weathering processes, probably mainly frost action, combined with the presence of water from springs and groundwater seepage.

The type and intensity of sliding varies as one proceeds upstream and this variation is related to the character of the underlying

geological formations.

5.2 Waterways Formation

This formation is not involved in any sliding or slumping.

5.3 McMurray Formation

No slides were noted in this formation, which under the present conditions stands on steep uniform slopes.

5.4 Clearwater Formation

This formation is involved in numerous shallow slumps which develop into flow slides. Sliding in the Clearwater Formation is closely related either to springs which appear at the contact with the overlying Grand Rapids sandstone, or to minor seepages within the sand horizons of the Clearwater Formation itself. Spring or seepage water softens and saturates the formation, causing the shallow slumps and resulting flow slides.

Although instability is common throughout the area underlain by the Clearwater Formation, it appears to be less frequent upstream of Brule Rapids, about 30 miles upstream from Crooked Rapids. This may be related to a change in character of the formation, or to the presence of the Joli Fou Formation as discussed in 5.5 below.

5.5 Grand Rapids Formation

There are many slides, of a distinctive type, in this formation The Grand Rapids sands are relatively pervious and, as discussed above, springs emerge at the contact with the underlying Clearwater

Formation. Springs also appear to emerge at the top of cemented impervious horizons within the Grand Rapids formation itself. These springs erode the uncemented sand and also saturate the resulting debris, which then flows down the slope leaving a steep cliff, which appears to progressively fail as the process continues. These cliffs form prominent amphitheatres which can be seen from a few miles downstream of Crooked Rapids to near Brule Rapids. Upstream from a location about 7 miles downstream of Brule Rapids, the Grand Rapids Formation is overlain by the Joli Fou shale. This shale appears to protect the Grand Rapids Formation from upland seepage thus alleviating the sliding within it.

5.6 Joli Fou Formation

As stated above, this formation first appears about 7 miles downstream of Brule Rapids and is therefore not mentioned in Section 4, Damsite Geology, nor was it examined on the ground. From the literature it is known that the Joli Fou Formation is a relatively young clay shale and it appears to be unstable even on moderate slopes. It is likely that where this formation becomes thicker, i.e. upstream of the area examined, the slides within it will become much more deep-seated than those in the other formations discussed above.

6. CONSTRUCTION MATERIALS

6.1 General

The results of the search for construction materials, for fill and concrete aggregate, are given in detail in Appendix D.

The investigations were carried out in three phases:

a. First a review was made of what was already known regarding availability of construction materials in the general area.

b. An airphoto study was then carried out.

c. Exploratory drilling was then performed at locations selected on the basis of Phases a. and b.

The areas investigated and the extent of the investigation are shown on Fig. 5. It will be noted from this figure that of the three areas investigated one, Borrow Search Area 3, was given only a general appraisal by means of a rather superficial airphoto study, while the other two areas, Borrow Search Areas 1 and 2, were subjected to a more detailed airphoto study, and exploratory drilling was carried out within them.

Area 1 was chosen to be investigated in more detail than Area 3 not because it was considered likely that it would contain better construction materials. Rather, it was selected on the basis that a study of the literature, and a general airphoto study, indicated that the materials likely to be encountered for a considerable distance on either side of the Athabasca River from Fort McMurray to at least 100 miles upstream were likely to be similar, and that the investig-

ation should therefore be concentrated close to the postulated damsites. The literature and airphoto studies indicated that it was unlikely that large quantities of good construction materials would be found in Area 1, and, as will be indicated below, this in fact turned out to be the case. As a result, the investigation was directed to Area 2, which, although over 25 miles from the postulated damsites, was known to contain certain deposits which were considered worth investigating.

6.2 Borrow Search Area 1

Forty two test holes were drilled in Area 1, using a helicopter-transported auger drill.

The area is, in general, covered by muskeg and windblown sand. The muskeg varies in thickness from a foot or two, to possibly over 15 ft. The windblown sand, which is a fine uniform sand, can be as much as 30-ft thick when it is in the form of dunes, but only a few feet thick when it is in sheet form.

In the northern quarter of the area the cover of muskeg or sand is immediately underlain by a deposit of glacial till which in one drill hole was found to a depth of 56 ft. This till can be described as a sandy silty clay with a trace of fine gravel. The material is generally of low to medium plasticity. At its natural water content it has a soft to stiff consistency with a minimum water content at least 6% above Standard Proctor optimum.

In the southern three-quarters of the area the cover of muskeg or sand is immediately underlain by glacio-lacustrine sandy silt and clay. This in turn, over at least part of the area, is underlain by

the glacial till described above. The glacio-lacustrine material occasionally contains ice-rafted pebbles, making it difficult to distinguish it from the glacial till. Like the glacial till the lacustrine sandy silt and clay is generally low to medium plasticity, with a natural moisture content very considerably above Standard Proctor optimum.

In Area 1, along the Athabasca River itself, there are a few small fluvial terraces of sand and gravel. By far the major terrace in the area occurs about a mile downstream from the top of the Crooked Rapids meander, where a 25-ft thick face of bouldery gravel with sand is exposed. It is not known how much material is contained in the terrace since the deposit could not be drilled with the auger equipment being used in the investigation, but it is possible that it could contain up to a total of ½-million cu yd.

6.3 Borrow Search Area 2

Nine test holes were drilled in Area 2 using a track-mounted auger drill.

One hole encountered glacial till right from the surface, while another six holes encountered till below a depth of sand varying from 2 to 44 ft. The maximum depth to which till was encountered was 50 ft. The till is similar in appearance and properties to the till encountered in Area 1, although there is an indication that at some locations it may be marginally drier near the top of the deposit.

Two holes were drilled into a kame moraine recognized from airphotos. One of these holes encountered sand and gravel throughout its 23-ft depth, while the other encountered

poorly-graded sand to a depth of 30 ft where it penetrated the glacial till described above. On the basis of these test holes, on visual inspection of Department of Highways' sand and gravel borrow pits in the area, and on a knowledge of the geology, it is believed that the kame deposit might in general be some 30 to 50-ft deep, and of a variable nature, with some of it all sand, and some a stratified mixture of sand and gravel. It is likely that the sand will predominate. The airphoto study indicates that the kame fields could cover an area of about 80 million sq yd.

Six holes were drilled through glacial outwash sands which had been deposited against the northwest edge of the kame moraine discussed above. The outwash is a fine to medium sand with a small amount of gravel. It is underlain by the glacial till at depths varying from 2 ft to over 40 ft. On the basis of the exploratory drilling, on visual inspection of Department of Highways' borrow pits, and on a knowledge of the geology, it is believed that the average depth of the outwash sand deposits might be some 35 ft. The airphoto study indicates that the deposit could cover an area of about 20 million sq yd.

6.4 Borrow Search Area 3

As stated above it is believed that the materials in Area 3 are similar to those encountered in Area 1. An exception to this is the presence of significantly-sized outwash sand and gravel deposits near Waterways, at least one of which is presently being used for granular borrow. These outwash sand and gravel deposits are some 25 miles from the postulated damsites, and considering their importance to the developments in the vicinity of Fort McMurray,

they were not examined in any detail in connection with this study.

6.5 Required Excavations and Quarries

The investigations described above clearly showed that there is a great scarcity of good borrow materials in the vicinity of the postulated damsites. This indicated that the use of materials from the required excavations and from quarries, particularly in the Devonian limestone, would have to be considered. However time and budgetary restraints did not permit exploratory drilling to be carried out to this end and an assessment of the suitability of such materials had to be based on a knowledge of the Damsite Geology as given in Section 4. This is further discussed in Section 12. Potential quarry sites are shown on Fig. 2.

7. PROJECT LAYOUTS

7.1 General

It is clear from the information given in Sections 3 through 6 that there are major technical, and thus economic, problems connected with the postulated development at Crooked Rapids. However a detailed discussion of these, and of the assumptions made in arriving at the project layouts, will be reserved for Section 12. In this Section 7 the various main elements of the projects will simply be described, with the minimum of discussion as to why those elements were chosen.

7.2 Medium-Head Project

a. Layout

The general arrangement of the Medium-Head Project is shown on Fig. 6, while the sections through the dam and associated works are shown on Figs. 7 and 8.

It will be noted from these figures that the project is shown as comprising a 255-ft high earth/rock dam, 3 large-diameter diversion/power tunnels, a large chute spillway, and a 120 MW surface powerhouse. It will also be noted that deep excavations would be required, and that very extensive seepage control measures would be needed. The basic features of the various elements of the project are described below.

b. Dam

It is apparent from Section 4, Damsite Geology, that, apart

from any other considerations, the dam would have to be of earth, or a combination of earth and rock. The extremely poor abutment foundation conditions rule out a concrete dam of any type.

Plan and sectional views of the dam are shown on Figs. 6 and 7 respectively.

The location of the dam axis and the crest elevation of 1150 ft, giving about a 250-ft high dam with a 25-ft freeboard, were chosen to take maximum advantage of the topographic features of the site as discussed in Section 2 of the report.

The dam would have exterior slopes of 1 on 2¼ upstream and 1 on 2 downstream. It would be a zoned section with a centrally located impervious core of oil-rich tar sands, supported by wide transition zones of fine limestone rock or sandy material from the required excavations, and shells of quarry-run limestone. It would have internal filter and drainage zones of processed gravel, and limestone rock slope protection. The 85-ft high upstream, and 15-ft high downstream cofferdams would be largely incorporated into the dam section. This dam section was chosen because of the scarcity of suitable more conventional borrow materials in the vicinity of the postulated damsite.

Most of the material in the dam would have to be placed in thin lifts and be very well compacted by heavy compaction equipment.

c. Tunnels

Plan and sectional views of the tunnels are shown on Figs. 6 and 8, respectively. All 3 tunnels would be used initially during construction as free-flow river diversion tunnels. A maximum diversion flow of 200,000 cfs, equivalent to a flood with an estimated return frequency of about 100 years, allowing for some storage behind the upstream cofferdam (see Fig. 3), was used to compute the required number and diameter of the tunnels. In order to carry the design diversion flow the tunnels would have to be 41-ft diameter after lining. Although the bottom portion of the tunnel excavation would be in limestone, which as was discussed in Section 4 is relatively competent, above the spring line the excavation would be in the tar sands. An allowance for a substantial thickness of lining has therefore to be made and the excavated diameter of the tunnels would be about 50 ft. Owing to the relatively flat slope of the existing ground, the tunnels would have cut and cover sections at both ends.

After all 3 tunnels were no longer needed for river diversion one of them would be converted to a power tunnel. This would be accomplished by installing gates through a concrete plug about midway along and by constructing a 25-ft diameter steel penstock in the downstream half of the tunnel. The gates, one control gate and one service gate, would be operated from an underground chamber to which access would be obtained from the adjacent diversion tunnel. It might also be convenient to provide the gate atmospheric air supply from this tunnel.

The intake to the power tunnel would be through a tower constructed at the upstream end of the tunnel. This tower

which would be founded on limestone, would house a movable hemi-spherical bulkhead. The bulkhead would be in the closed position while the concrete plug shown downstream of the closure gate was being constructed. It would then be raised so that water could enter through the trash-rack openings. Provision would be made to allow the bulkhead to be lowered at any time so that the upstream portion of the tunnel could be inspected after the power plant was in operation.

d. Spillway

It will be noted from Fig. 3 that the estimated Maximum Probable Flood is 570,000 cfs. Flood routing studies indicate that even allowing for a reservoir surcharge of 10-ft above Normal Full Pool the amount of storage is insufficient to make the spillway design flood significantly less than the Maximum Probable Flood. In order to cater for a design flood of this magnitude a chute spillway would be required. A tunnel spillway can be ruled out. The spillway shown has thus been designed to pass 570,000 cfs when the reservoir is 10-ft above Normal Full Pool.

Plan and sectional views of the spillway are shown on Figs. 6 and 8, respectively. The spillway chute would be 300-ft wide and the flow controlled by means of 6 - 50-ft by 50-ft radial gates.

Because of the extremely poor rock conditions, the chute would have to have extensive underdrainage, and the flip

bucket would have to be well keyed into limestone which had been thoroughly consolidated by grouting. For the same reason the approach channel would have to be lined with concrete or possibly soil cement, and the lining well protected by underdrainage.

The chute and flip bucket would have to be designed to ensure that the plunge pool which would develop would not undermine the flip bucket.

e. Powerhouse

Plan and sectional views of the powerhouse are shown on Figs. 6 and 8, respectively.

The powerhouse would be a surface structure founded well into limestone.

It would be served by three $14\frac{1}{2}$-ft diameter unit penstocks resulting from a trifurcation of the 25-ft diameter penstock described in c. above.

The structure would house, as will be discussed in Section 9, three 40 MW generators served by three 55,000 hp Francis turbines.

f. Outlet Works

As will be noted from a comparison of Fig. 1 and Table 1 (Section 3), the storage capacity of the Medium-Head Dam is small compared with the natural river flows in all but the winter months. As a result, the spillway would be discharging for most of the year. It would not appear to be necessary, therefore, to have a separate outlet works since the spillway discharges should be more than adequate to meet riparian requirements.

g. Switchyard

The switchyard would be located on the same bank as the powerhouse, between the powerhouse and the downstream toe of the dam.

h. Excavations

Again because of the extremely poor rock conditions, all permanent excavations for structures, except excavations made in the limestone, would have to have flat slopes. Furthermore, because of the inherent instability of the valley walls, as discussed in Section 5, excavations to flat slopes would have to be made for some considerable distance upstream and downstream of the dam.

The extent of the required excavations can be seen from Figs. 6, 7 and 8. Essentially, all permanent excavation slopes in the vicinity of structures have been shown as 1 on 3, while the others have been shown on either 1 on 3 or 1 on $2\frac{1}{2}$, depending upon whether the excavation is subjected to reservoir drawdown or not.

Generally, where the overall existing ground is lying at a slope flatter than 1 on 3 no excavation has been indicated. However downstream on the right bank some excavation would in fact have to be carried out to remove locally unstable material.

i. Seepage Control

The rock conditions, particularly the presence of uncemented sands and silts, make it imperative to ensure that all

seepage from the reservoir into the abutments would be strictly controlled. To this end an extensive seepage control scheme is shown on Figs. 6, 7 and 8.

The first element in this scheme, but probably the least important, is the grout curtain. This grout curtain would be confined to the limestone. It is shown as a double line curtain to a depth equal to half the reservoir head. However this is a rather arbitrary assumption since nothing is known about the permeability of the limestone at the postulated site.

The second element in the seepage control scheme is the ICOS-type wall. This underground cut-off wall is shown as extending up from the top of the limestone to the bottom of a 50-ft deep cut-off trench which would be excavated along the line of the impervious core of the dam, past the spillway on the right bank and well into the abutment on the left. The 2-ft thick concrete wall would be constructed from the bottom of the trench by means of an excavation and slurry - replacement procedure developed by I.C.O.S. of Milan, Italy. After completion of the wall, the trench, or at least the central part of it, would be backfilled with impervious material.

The third and final element of the seepage control scheme would be a series of 7 ft by 9 ft concrete lined drainage adits. The adits would have drain holes drilled up from them, and because of the nature of the rock these holes would have to contain perforated pipes and filters. The drainage scheme would be designed to collect water which would seep around the cut-off wall and to remove this water in a controlled fashion, thus protecting the dam, the spillway and the downstream abutment slopes.

7.3 High-Head Project

a. Layout

The general arrangement of the High-Head Project is shown on Fig. 9, while the sections through the dam and associated works are shown on Figs. 10 and 11.

It will be noted from these figures that the project is shown as comprising a 540-ft high earth/rock dam, 3 large diameter diversion/power tunnels, a large chute spillway, and a 560 MW surface powerhouse. It will also be noted that deep excavations would be required, and that very extensive seepage control measures would be needed.

The basic features of the various elements of the project are discussed below.

b. Dam

As in the case of the Medium-Head Project, the abutment foundation conditions alone dictate that the High-Head Dam would have to be of earth or earth/rock construction.

Plan and sectional views of the dam are shown on Figs. 9 and 10, respectively.

The crest of 1450, which gives a freeboard of 30 ft, was chosen so that the dam would be as high as the topography would allow, bearing in mind that up to 50 ft of overburden could exist at the site. The location of the dam axis

was chosen such that on the left bank the dam was a safe distance from the slope cut by the ancient meander discussed in Section 4 (see Photo 1) without increasing the length of the spillway chute excessively, while on the right bank the dam would enter a gully and buttress a potentially unstable side slope.

The dam would have exterior slopes of 1 on $2\frac{1}{2}$ upstream and 1 on $2\frac{1}{4}$ downstream. It would have a similar zoned section and would be constructed of essentially the same materials as the Medium-Head Dam. The 108-ft high upstream and 30-ft high downstream cofferdams would be largely incorporated into the dam section.

c. Tunnels

Plan and sectional views of the tunnels are shown on Figs. 9 and 11, respectively.

As in the case of the Medium-Head Project all three tunnels would be used initially as free flow tunnels for river diversion, and the finished diameter of 41 ft, the same as in the Medium-Head Project since the maximum diversion flow would be the same, was chosen on this basis. In this case, however, it would appear that the tunnels would be excavated entirely in the Tar Sands which would mean more difficult construction and probably a somewhat thicker lining than in the case of the Medium-Head Project. Again owing to the relatively flat slope of the existing ground, the tunnels would have cut and cover sections at both ends.

After all three tunnels were no longer needed for river diversion, two of them would be converted to power tunnels. This would be accomplished in a similar manner to that described for the Medium-Head Project, but the penstocks in this case would be 23 ft rather than 25 ft in diameter.

The gates, gate chamber and power intakes would be as described for the Medium-Head Dam.

d. Spillway

Plan and sectional views of the spillway are shown on Figs. 9 and 11, respectively.

Unlike the Medium-Head Dam, flood routing studies indicate that if 10 ft of surcharge above Normal Full Pool is allowed for, the storage capacity of the High-Head Dam is such that the spillway design flood can be made considerably less than the Maximum Probable Flood. The studies show that a spillway designed to discharge 475,000 cfs when the reservoir is surcharged by 10 ft would be adequate to pass the Maximum Probable Flood. This figure is based on the assumption that the flood would enter a reservoir which was at Normal Full Pool level, and that the gates would be operated to maintain the reservoir at this level as long as this was possible. When the gates were all fully open, the reservoir level would then rise to a maximum of 10 ft above Normal Full Pool.

Although it would have a smaller crest width than the Medium-Head Project spillway, the design and layout of the High-Head Project spillway would be similar to that of the Medium-Head

Project spillway and as a result comments in 7.2 d. apply in this case also.

e. Powerhouse

Plan and sectional views of the powerhouse are shown on Figs. 9 and 11, respectively.

The powerhouse would be a surface powerhouse founded well into limestone.

It would be served by four 16-ft diameter unit penstocks resulting from a bifurcation of each of the 23-ft diameter penstocks described in 7.3c.

The structure would house, as will be discussed in Section 9, four 140 MW generators served by four 192,000 hp Francis turbines. The turbines, as will be discussed below, would be fitted with energy dissipating bypass valves.

f. Outlet Works

Unlike the situation with the Medium-Head Project, the storage capacity of the High-Head Project reservoir is such that in a low flow year it is possible that, as will be noted in Section 9, the spillway would not be in operation. However, the firm regulated outflow from the powerhouse in the critical dry period is 13,530 cfs. This outflow must be about equal to the maximum riparian requirements (for navigation). Consequently to cater to the situation where the powerhouse is not in full operation it would be desirable to have a bypass in the powerhouse.

Although this bypass would result in a significant increase in the cost of the turbines, it would be less expensive then providing a separate outlet works.

g. Switchyard

The switchyard would be located on the same bank as the powerhouse, between the powerhouse and the downstream toe of the dam.

h. Excavations

The extent of the required excavations can be seen from Figs. 9, 10 and 11. These excavations would have to be made to slopes of 1 on 3 and 1 on $2\frac{1}{2}$, for the reasons discussed above for the Medium-Head Dam. The excavation shown on the left bank for some 2000 ft downstream of the dam would be carried out to remove existing slide scarps and debris (see Photo 3) and to ensure the stability of the slope above and downstream of the powerhouse.

Generally, where the overall existing ground is lying at a slope flatter than 1 on 3, no excavation has been indicated. However, under the dam and downstream on the right bank, and below and upstream of the spillway approach channel on the left bank, some, possibly quite considerable, excavation would in fact have to be carried out to remove locally unstable material.

i. Seepage Control

As in the case of the Medium-Head Project, and for the same reasons, extensive cut-off and drainage measures would have to be taken to control seepage. The extent of these measures are shown on Figs. 9, 10 and 11.

The discussion of the seepage control measures given in 7.2 i for the Medium-Head Project is directly applicable to the High-Head Project. However it will be noted that in the High-Head Project the ICOS-type wall would have to be constructed to much greater depths, and in fact, the maximum depth of the wall shown is beyond precedent. As a result, the cut-off trench from which the wall would be constructed would have to be made deeper than in the case of the Medium-Head Dam, probably up to 100 ft in depth.

8. COST ESTIMATE AND CONSTRUCTION SCHEDULE

8.1 General

Details of the capital cost estimate and the construction schedules for the two postulated projects are given in Appendix F.

8.2 Cost Estimate

A summary of the capital cost estimate is given in Table 2.

Table 2
Summary of Estimated Capital Cost
Medium-Head and High-Head Projects

Item	Amount	
	Medium-Head Project	High-Head Project
Construction Cost	$ 248,300,000	$ 625,800,000
Contingency (15%)	37,200,000	93,900,000
Investigation, Engineering and Supervision (10%)	28,600,000	72,000,000
Interest During Construction (at 9% p.a.)	84,600,000	327,700,000
TOTAL CAPITAL COST	$ 398,700,000	$ 1,119,400,000

It should be noted that no allowance was made in the estimate for escalation. Consequently the total capital cost is given in January 1975 dollars.

The 9% p.a. used for Interest During Construction is the approximate long term interest rate currently available to government borrowers.

It should also be noted that the estimate does not include transmission costs.

8.3 Construction Schedule

It is estimated that the total construction times for the postulated Medium-Head and High-Head Projects are 5 years and 7 years, respectively.

9. POWER STUDIES

9.1 General

The results of the power studies for both postulated projects are given in detail in Appendix G.

For each dam the following were calculated:-

a. the firm outflow based on the most critical dry period, and the corresponding firm energy, and

b. the total annual energy which could be generated in the average flow year at plant capacity factors of up to 100%, when water was available. The installed capacities were determined by applying assumed capacity factors of 50% and 75% to the firm energy generated in the critical dry period.

In accordance with the Terms of Reference discussed in Section 2 the power study was carried out assuming that each dam would be the first development on the river. It was also carried out without knowledge of the characteristics of the power system with which the new plant would have to be co-ordinated and no assumption was made regarding the position of the new plant on a load curve.

It was further assumed in the power study that the reservoir impounded by each postulated dam would be operated for optimum power generation, no consideration being given to such factors as flood control or navigation, items which will be discussed in Sections 11 and 12.

9.2 Basic Data and Assumptions

The basic data used in the power study were the mean monthly flows given in Table 1 (Section 3), the reservoir volume curves shown on Figs. 6 and 9, and the tailwater rating curves shown on Fig. 2.

The basic assumptions used in the study are as follows:-

	Medium-Head Project	High-Head Project
Normal Full Pool Elevation	1125 ft	1420 ft
Minimum Pool Elevation	1050 ft	1253 ft
Available Live Storage	48×10^4 ac-ft (Fig. 6)	391×10^4 ac-ft (Fig. 9)

For both dams the natural mean monthly flows were taken from Table 1, and an average flow year was taken as the set of 12 successive 60-year mean monthly flows given in this table. An overall plant efficiency factor of about 85% was adopted.

9.3 Results

A summary of the results of the power study is given in Table 3.

On the basis of the results given in Table 3 it has been assumed that the installed capacities at the Medium-Head and High-Head Projects would be 120 MW and 560 MW, respectively. The choice of these installed capacities is somewhat arbitrary since as was stated above, the power study was carried out without knowledge of the characteristics of the power system with which

the Crooked Rapids plant would have to be co-ordinated. However it is clear that in the case of the Medium-Head Project, with its very limited storage, the installed capacity should be well in excess of the firm power output of 60 MW. A plant with a capacity of 120 MW could be utilized to its full for 7 months in the average flow year and such an installation could produce total energy 1.63 times the firm energy. It would thus appear reasonable to assume an installed capacity of 120 MW at this time. The final capacity decided upon would depend upon system requirements and costs of alternatives.

In the case of the High-Head Project the choice of such a high rate of installed capacity to firm output probably cannot be justified. For example an installation of 840 MW could be utilized to its full for only 2 months in an average flow year and would produce total energy only 1.31 times the firm energy. A plant with a capacity of 560 MW would on the other hand be utilized to its full, or close to it, for 6 months in the average flow year, and would produce total energy 1.18 times the firm energy. It would appear reasonable, therefore, to assume an installed capacity of 560 MW at this time. However, again, the final capacity decided upon would depend upon the system requirements and the cost of alternatives.

TABLE 3

Summary of Power Study

Medium-Head and High-Head Projects

Item	Medium-Head	High-Head
Firm Outflow	4300 cfs	13,530 cfs
Firm Power	60 average MW	420 average MW
Firm Annual Energy	526 GWh	3,679 GWh
Annual Energy Output in Average Flow Year		
Installed Capacity		
80 MW	663 GWh	
120 MW	860 GWh	
560 MW		4,334 GWh
840 MW		4,825 GWh

9.4 Energy Costs

a. Annual Charges

On the basis of a study of the methods used by British Columbia Hydro and Manitoba Hydro to estimate their annual charges for hydroelectric projects the following approach was taken. This approach is rather simplified but it is believed adequate in view of the preliminary nature of the study. The various annual charges are given as percentages of the total capital cost.

Service Life	60 years
Fixed Charges	
Interest	9.00%
Sinking Fund	0.05%
Operation and Maintenance	0.60%
Administration and General	0.20%
Insurance	0.15%
TOTAL ANNUAL CHARGES	10.00%

On the basis of the capital cost estimate given in Section 8, the annual charges for the Medium-Head and High-Head Projects would thus amount to $39,900,000 and $111,900,000 respectively. It should be noted that these charges do not include transmission costs.

b. Costs

The estimated energy costs for the two postulated projects are given in Table 4.

TABLE 4

Estimated Energy Costs

Medium-Head and High-Head Projects

Item	Medium-Head	High Head
Installed Capacity	120 MW	560 MW
Annual Firm Energy Output	526 GWh	3679 GWh
Average Annual Energy Output	860 GWh	4334 GWh
Total Capital Cost	$398,700,000	$1,119,400,000
Annual Charges	$ 39,900,000	$ 111,900,000
Cost/Installed kW	$ 3,320	$ 2,000
Cost/Firm kWh	76 mills	30 mills
Cost/Total kWh	46 mills	26 mills

It should be noted that all costs given in Table 4 are for power at site. No transmission costs are included. It should also be noted that although an installation of less than 120 MW would be needed in order to exploit the firm energy only, the effect of this reduced installation on the mill rate would be negligible owing to the very high capital expenditures involved in the diversion works, the dam, and the spillway, which are independent of the installed capacity.

10. RESERVOIR SEDIMENTATION

10.1 General

The results of the reservoir sedimentation study are given in detail in Appendix H.

The sedimentation study was carried out even at this preliminary stage of the Feasibility Study since suspended sediment measurements being collected on the Athabasca River at Fort McMurray indicated that large sediment loads were being carried by the river. Excessive sediment accumulation in the reservoir could have an adverse effect on the ability of a dam at Crooked Rapids to store water in the long-term.

10.2 Method

The sedimentation study was carried out as follows:

a. First, the suspended sediment records available for the Athabasca and Clearwater Rivers near Fort McMurray were analyzed and an annual sediment load versus river discharge relationship for the Athabasca River was developed.

b. An annual flow duration curve as derived from Table 1 (Section 3) was then used in combination with the sediment load curve to obtain an estimate of the average annual suspended load which would enter the postulated reservoir.

c. An adjustment to this value for bed load transport was then made to give the total sediment load which would enter the reservoir.

d. Finally, a study was made of the probable range in trap efficiencies of the reservoir on the basis of probable reservoir operating levels. This, combined with an assumed density of the sediment, enabled the rate of sediment build-up to be estimated.

10.3 Results

The results of the reservoir sedimentation study are given in Table 5. This table is the end result of the computations made to predict the time required to fill the two reservoirs with sediment at the estimated realistic range of trap efficiences.

Table 5
Reservoir Sediment Accumulation Times

Project	Trap Efficiency %	Time to Fill Reservoir (years)
Medium-Head	70	112
	89	88
High-Head	91	630
	99	578

11. ICE CONDITIONS

11.1 General

The results of the ice study are given in detail in Appendix I.

The study was carried out to assess the impact of the postulated development at Crooked Rapids on the ice cover of the Athabasca River downstream.

Important considerations in the study included a prediction of the length of open water which would be sustained downstream of the dam because of wintertime releases from the reservoir, as well as the effect of any change of the ice cover on the springtime break-up process. The latter consideration is important to the town of Fort McMurray because of flooding problems experienced in the past during break-up.

11.2 Approach

a. Field

Visits to the relevant stretch of river were made during the spring break-up and fall freeze-up periods in 1974. A comprehensive photographic record was made of the 1974 break-up and this has been presented separately to the Department.

b. Office

The office study was carried out as follows:

i First, average winter reservoir temperatures were estimated on the basis of the monthly flows given in Table 1 and on the reservoir volumes shown on Figs. 6 and 9.

ii It was then assumed that the outflow temperature would be equal to the computed average reservoir temperature.

iii The length of ice-free river below the dam was then estimated on the basis of accepted heat dissipation formulae.

11.3 Results

The basic results of the ice study are:

a. Medium-Head Project

Because of the low winter discharges from the Medium-Head Project a relatively stable ice cover would still develop upstream from Fort McMurray in an average flow year. Some local flooding would occur near the upstream end of this ice cover but this would only affect Fort McMurray during abnormally warm periods between January and March. During such periods, water level fluctuations of possibly 10 ft would occur near the upstream end of the cover.

b. High-Head Project

Because of the greater discharges from the High-Head Dam,

in an average year open-water would be maintained to a point downstream of Fort McMurray, except for a brief period in February. Even in February, this ice cover would tend to be broken by the mechanical action of a fluctuating tailwater at the dam. Nevertheless, in February some local flooding, involving water level changes of possibly 10 ft, could occur at the upstream end of the cover. During the remainder of the year, when the ice-free area would extend from the dam to below Fort McMurray, daily level fluctuations at the upstream end of the cover would be reduced to possibly 5 ft, with a consequent reduction in minor winter flooding.

Since in an average year the ice-free reach would extend below Fort McMurray, after February the Clearwater River would be able to discharge its ice freely in most years, thus preventing the type of flooding which occurred at Fort McMurray in April 1974.

c. Operation

Because of the slow rate of reduction in ice-free distance during freeze-up, it is unlikely that any benefit would be realized by restricting the operation of either the Medium-Head or High-Head Projects during the freeze-up period. Furthermore, it is unlikely that the normal operating schedule would need to be altered during break-up, since the maximum estimated rates of flow from both the High-Head and Medium-Head Projects would be well below flow peaks which now occur naturally because of jams breaking up in the area which would be flooded by the reservoir.

In addition, the volume of ice available to form a jam in the vicinity of Fort McMurray would be greatly reduced by the presence of either the High-Head or Medium-Head Dam. Because of these factors, the ice cover could be expected to remain relatively intact for a longer period than under natural conditions, and thus to be more weakened by the effect of solar radiation when break-up does occur.

Normal winter activity on the ice cover within about 10 miles of its upstream edge will be considerably less safe than under natural conditions because of minor flooding caused by flow and weather changes.

12. DISCUSSION

12.1 General

It is clear from the previous sections of the report that there are severe technical problems associated with any postulated hydro development at Crooked Rapids. In this Section 12 these problems are discussed. In addition, a brief mention is made of some of the effects of a development at Crooked Rapids on the areas upstream and downstream of the dam.

12.2 Seepage Control

a. Abutments

As will be noted from Section 4, the geological strata above river level at Crooked Rapids contain large thicknesses of uncemented silt and fine to medium grained sand. In fact about 90 per cent of the Grand Rapids Formation and possibly 60 per cent of the Clearwater Formation fall into this category. Seepage through such materials would have to be prevented in the vicinity of the dam. Otherwise internal erosion, or piping, could occur with very serious consequences.

The grain sizes and densities of the silt and sand members are such that it would not appear to be practicable to grout them effectively, even with a chemical grout. As one alternative to grouting, the use of a long upstream blanket of impervious material was considered. However the design and construction problems involved in the use of such a blanket are so great that, like grouting, this

approach was ruled out. The approach adopted for the project layouts shown is the use of a positive cut-off in the form of an ICOS-type wall (An ICOS wall is an underground concrete wall which is constructed by excavating a narrow trench, the sides of which are stabilized during excavation by means of a thick mud slurry. The slurry is then displaced by tremie concrete).

In the project layouts shown on Figs. 6 through 11, in order to keep the depth to a minimum it was assumed that the ICOS-type wall would be constructed from the bottom of a deep cut-off trench, and that the wall would extend from the bottom of this trench down to the top of the limestone. The top of the limestone is known to be some 20 ft above river level at the site of the postulated Medium-Head Dam, and it is believed (but not known) to be about riverbed level at the site of the postulated High-Head Dam. On this basis, the maximum depth of wall which would have to be constructed at the Medium-Head and High-Head sites would be about 200 ft and 480 ft, respectively. The maximum depth to which such a wall has been constructed is 420 ft, and it becomes a more difficult and slower operation once the depth exceeds 200 to 250 ft, (since the type of excavation tool has to be changed). It is clear, therefore, that, if the cut-off wall had to be taken down to tie into the limestone, the operation would be relatively straightforward for the Medium-Head, but extremely difficult for the High-Head Dam.

Although it cannot be assumed at this time, it is possible that the bottom 80 or 100 ft of the oil-rich Middle Member

of the McMurray Formation, i.e. the Tar Sands Formation overlying the limestone, is sufficiently impervious and stable enough against piping that the ICOS-type wall could be terminated within it. This would decrease the depth of the wall by up to 100 ft and would thus make construction of a cut-off for the Medium-Head Dam fairly simple. It would, of course, also improve considerably the situation for the High-Head Dam, but the task would still be a most difficult one.

On the other hand, it is possible that after investigation it would be decided that a single cut-off wall could not be relied upon and that a double wall was required. It could also be decided that the wall should be extended further into the abutments than is presently shown.

It will be noted from Figs. 6 through 11 that, in addition to the cut-off wall, a comprehensive system of drainage curtains would be needed to collect any seepage through and around the ends of the wall, and to remove the water in a controlled manner.

The drainage curtains would consist of a series of adits from which upward drain holes would be drilled at about 10 ft centres. To eliminate the possibility of piping of silt and sand into the drain holes, it is likely that the holes would have to contain perforated pipes and filters, a difficult and expensive job. Furthermore, some of the adits for the High-Head Project would be excavated mainly through uncemented sands. This would be no easy task, especially

if water-bearing strata were encountered.

b. Limestone

Limestone often presents seepage problems in the form of solution channels and caverns. However at Crooked Rapids the limestone is very argillaceous and it might in fact be quite impervious. Nevertheless this would have to be proved, and until such times as drill hole water and grout tests are made substantial underseepage problems and dewatering problems (for example in excavations for the powerhouse and in quarry excavations) must be considered as possibilities.

c. Reservoir

Some water would probably seep through the sandy materials in the reservoir slopes. However, because of the long seepage paths it is likely that the water loss would be small compared with the inflow. Nevertheless it is a factor which would have to be checked once more is known about the extent and permeability of the various geological formations.

12.3 Slope Stability

a. General

It will be noted from Section 5, that there is conspicious evidence of present and past slope instability all along the Athabasca River Valley. In the general vicinity of the postulated damsites, the geological members which are, or have been, involved in sliding are the Grand Rapids and Clearwater Formations.

b. Immediate Vicinity of Damsite

As pointed out in Section 5, the presence of seepage water and springs appears to be the basic cause of instability. Consequently, the presence of a dam would tend to make the situation worse, especially upstream, where the slopes would be subject to reservoir drawdown. Even downstream of the dam, precautions would have to be taken, since some seepage from the uplands and from the reservoir could still bypass the seepage control measures described above.

The slope instability problem in the immediate vicinity of the damsite, where slope failures could undermine or otherwise damage project structures, has been taken into consideration in the project layouts shown in Figs. 6 through 11 by ensuring that all cuts would be excavated to flat slopes. The slopes chosen, and these would have to be confirmed by extensive investigation, are 1 on 3 adjacent to structures and where the slope would be subject to reservoir drawdown, and 1 on $2\frac{1}{2}$ elsewhere.

One of the factors which would have to be thoroughly investigated in connection with slope stability is the extent and detailed physical properties of the layer of highly plastic clay and clay shale which, as described in Section 4, was found at the top of the Clearwater Formation at Geological Section S1. It is possible that a detailed study of this layer would lead to a further flattening

of the slope upstream of the High-Head Dam, especially in the vicinity of the spillway approach channel.

c. Reservoir

It was pointed out in Section 5 that there are at present numerous slides in the reservoir area but that these are shallow-seated (except possibly in the Joli Fou shale which enters the geological column too far upstream of the damsites covered by this report to be of concern). There is no doubt that, if a dam were to be constructed at Crooked Rapids, the number of slides in the reservoir area would greatly increase. It is also very likely that larger volumes of material would be involved than at present. Nevertheless, providing precautions were taken in the immediate vicinity of the damsite, as described above, and providing ample dam freeboard was allowed for, it is tentatively concluded that the dam would not be endangered by waves created by slides into the reservoir. The situation would of course, have to be thoroughly investigated.

The susceptibility of the reservoir slopes to sliding would also have a detrimental effect on the land use around the reservoir, and possibly on the use of the reservoir itself for recreation purposes.

d. Downstream

Landslides of limited size as described in Section 5, would continue to occur downstream of the dam. However,

it is unlikely that any change in river regime brought about by construction of the dam would exaggerate the situation to any significant degree. Consequently there would not appear to be a danger of landslides blocking the river downstream.

12.4 Construction Materials

a. General

As was pointed out in Section 6, large quantities of good construction materials could not be obtained from any borrow areas developed close to the postulated damsites.

The closest source of sand and gravel in any quantity appears to be over 25 miles from Crooked Rapids, in Borrow Search Area 2 shown on Fig. 5. Even in this area, sand may predominate, making the winning of gravel an expensive task. In fact, considerably more investigation would have to be carried out before it could be established whether gravel is present in sufficient quantities for the postulated projects.

Regarding impervious material, all the glacial till and lacustrine silts and clays encountered during the borrow investigation have natural water contents considerably in excess of those suitable for use in dam construction. Furthermore, the tasks of developing borrow pits in these materials and allowing them to dry out in the climate at Crooked Rapids are formidable.

b. Dam Fill

In light of the above, it was assumed that the dam would be constructed mainly of locally available rock materials. It will be noted from Figs. 7 and 10 that the sections chosen would consist mainly of locally quarried rock and sandy material from the required excavations.

It has been assumed that the impervious member in the dam could be oil-rich sand from the McMurray Formation, the Athabasca Tar Sands. While this is a reasonable assumption at this time such use for the Tar Sands is unprecedented. It must be emphasized, therefore, that a great deal of investigation would have to be carried out to determine the engineering properties and confirm the acceptability of the Tar Sands for this purpose. If the Tar Sands were to be used, they would be obtained from the nose of rock forming the Crooked Rapids meander where there is little cover of other formations over the McMurray Formation, and/or from the other potential quarry sites shown on Fig. 2.

The material for the bulk of the shells of the dam sections shown on Figs. 7 and 10 would be quarried limestone. It has been assumed at this time that the limestone at the site would not require a great deal of processing, and that the finer, dirtier, rock could be routed toward the centre of the section while the larger, cleaner, rock could be routed more towards the exterior slopes. It has also been assumed that limestone quarries would yield enough large rock to provide riprap and downstream slope protection. Again, while these assumptions are not unreasonable, a great deal of

investigation would have to be carried out to confirm their validity. The engineering properties of the limestone would have to be carefully evaluated since it is known that the material is fragmental and argillaceous, and that shale beds are present in the formations. It should also be pointed out that, although, as discussed in Section 4, the limestone member exposed at Crooked Rapids is the Moberly Member of the Waterways Formation, it is possible that, in the vicinity of the High-Head site this member is replaced by the Mildred Member, which may contain a higher percentage of shale beds. Potential limestone quarry sites are shown on Fig. 2 and cost studies indicate that it would be more economical to exploit these quarries than to haul sand and gravel from Borrow Search Area 2, even if it were proved that there was sufficient, fairly readily accessible, sand and gravel present there.

A most important zone in the dam sections shown on Figs. 7 and 10 is Zone 3, clean filter and drainage material. This material would have to be washed and screened and the individual particles would have to be most competent. In the cost estimates it was assumed that enough sand and gravel could be obtained from local river terraces to satisfy the Medium-Head Dam requirements. However in the case of the High-Head Dam it was assumed that sand and gravel would have to be brought in, probably from Borrow Search Area 2.

It will be noted from Figs. 7 and 10 that one of the two materials which would be allowed in Zone 2 is sandy material from the required excavations. Much of the required excavations would be through fine sand. However

much would also be through silts and shales. While a controlled amount of fine sand could be incorporated into the section, most of the silts and shales might not be acceptable. It is not possible to determine at this stage what percentage of the required excavations could be used from a technical point of view. Similarly, bearing in mind the complicated construction schedule and the fact that quarry waste should be kept to a minimum, it is not possible to evaluate how much of the required excavations it would be economical to use. For the cost estimate it was assumed that most of Zone 2 would come from quarries. However, considering the large quantities of material in the required excavations, this point would have to thoroughly investigated.

c. Concrete Aggregate

In estimating the cost of concrete aggregates it was assumed that the coarse aggregate could be produced by processing quarried limestone. Again, while this assumption is not unreasonable, a great deal of investigation would have to be carried out to confirm its validity.

In estimating the cost of fine aggregate, it was assumed that this material would have to be brought in from Waterways or Borrow Search Area 2.

12.5 Underground Excavations

a. General

It will be noted from Figs. 7, 8, 10 and 11 that with the

exception of some of the drainage adits, which are discussed in Section 12.2a, the underground excavations would either be partly in limestone and partly in the Tar Sands, or wholly in the Tar Sands. It is unlikely that the limestone would present any major problems, but how the Tar Sands would behave is not known at this time. There is no doubt that special construction techniques would have to be developed to cope with excavation in the oil-rich sands, and that excavation through them would be expensive. However, providing timely support was given to the excavation, it is possible that the material would behave like a soft rock, and that adequate designs could be readily developed. Nevertheless, the potential problems in excavating large openings in this material should not be under- estimated and extensive investigation would have to be carried out before designs could be finalized.

b. Tunnels

As was discussed in Section 7, the tunnels would have to be about 50 ft excavated diameter. In the case of the Medium-Head Project the bottom half would be in limestone while the top half would be in the Tar Sands. The fact that support could be provided by limestone at the spring line would help considerably in the excavations. In the case of the High-Head Dam, almost the entire opening could be in the Tar Sands, thus presenting a potentially worse situation than with the Medium-Head Project.

It will be noted from Figs. 8 and 11 that unless deep temporary excavations were to be made, which should be

avoided if possible, the tunnelling would have to start with less cover over the openings than is normally desirable and the methods of achieving this would have to be thoroughly investigated.

It would also have to be carefully considered whether the 3 large diameter parallel tunnels could be as close together as they have been shown on Figs. 6, 7, 9 and 10, considering the abutment conditions.

c. Gate Chambers

It will be noted from Figs. 8 and 11 that the power tunnel control and service gates would be operated from an underground gate chamber to which access would be obtained from an adjacent original diversion tunnel. This approach was taken since it was considered important to avoid shaft excavation through the extremely poor Grand Rapids and Clearwater Formations.

The gate chambers required would be large underground openings and, like the tunnels discussed above, they would have to be thoroughly investigated before safe designs could be developed.

12.6 Spillway

The spillways which would be required for either of the postulated projects would be unusually large and they would be founded on some of the poorest foundation materials.

The spillway approach channel would have to be lined with concrete, or possibly soil-cement, to prevent erosion, and the lining would have to have extensive underdrainage to cater for reservoir drawdown conditions. Similarly the spillway chute would have to have a thick concrete lining, protected by extensive underdrainage. The drainage systems would have to be designed in such a way as to prevent them from freezing.

The spillway control structures for either project would probably be founded on the Clearwater Formation. This could mean an expensive design, especially in the case of the High-Head Project if the highly plastic clay and clay shale layer discussed in Section 4 is found to be extensive.

A spillway discharging some 500,000 cfs would develop a huge plunge pool. The possible undermining effects of this on the spillway flip bucket would have to be thoroughly looked into by means of model tests. Fortunately the flip bucket at either project could be founded well into limestone, so that it is not likely that a major problem exists.

12.7 Power Intakes

As stated in Section 12.5c, above, it was considered important to avoid the use of shafts. Consequently, in the layouts shown on Figs. 8 and 11 the intakes to the power tunnels are shown as being through a tower constructed at the upstream end of the tunnels. In the case of the Medium-Head Project this does not place the intake much below Minimum Pool, i.e., the level at which it would have to be even if an intake shaft

were used. However in the case of the High-Head Project the intake shown on Fig. 11 is over 200 ft below Minimum Pool. Even although the intake tower would be founded on limestone it would not be desirable to increase its height much more than is shown. Consequently in the case of the High-Head Project the intake would always be submerged under at least 200 ft of water. This fact would have to be considered in much more detail before a final decision could be made on its acceptability.

12.8 Penstocks

It will be noted from Figs. 8 and 11 that for both the postulated projects the penstocks would be large-diameter steel pipes supported on concrete pedestals within the downstream half of the original diversion tunnels. In the case of the High-Head Project there would be a large-diameter penstock within 2 of the diversion tunnels and each penstock would be bifurcated thus serving 4 units in the powerhouse. In the case of the Medium-Head Dam, however, only one larger diameter penstock is shown. This penstock would then be trifurcated to serve 3 units in the powerhouse. This is the most economical solution in terms of capital cost, but it would result in complete shutdown of the units if, for example, the control gates needed repair or adjustment. From the operating point of view, it might have been better to have shown a penstock in each of 2 tunnels with 2 larger, or 4 smaller, units. However this could only be decided upon when more was known about the use to which the Crooked Rapids powerplant would be put, and how it would be co-ordinated with the existing power system.

12.9 Regulation

It will be noted from Fig. 11, as discussed in Sections 7.3e and 7.3f, that each turbine in the High-Head Project would be fitted with a bypass valve for discharging riparian flows up to turbine full load quantity. This valve could provide the dual function of a turbine pressure relief valve to provide satisfactory regulation during load rejection without the need for slow governor timing and additional generator inertia, or reduced pressure tunnel velocities. The design criteria would thus become the allowable speed drop during block loading of the unit, which is generally a less onerous requirement, and even without a surge shaft would probably be acceptable.

In the case of the Medium-Head Project, as discussed in Section 7.2f, no bypass valves would be needed for riparian releases, and as a result the powerhouse arrangement shown on Fig. 8 does not provide for any. However further studies may indicate that such valves may in fact be needed for turbine regulation purposes. Preliminary studies show that without pressure relief valves or a surge shaft the Medium-Head Project units would still be suitable for frequency regulation on a large power system, i.e. a system having a minimum on-line generation capacity of at least 3 times the power potential of the Medium-Head plant.

12.10 Reservoir Sedimentation

It will be noted from Section 10 that on the basis of a limited study the Medium-Head and High-Head reservoirs would silt up completely in periods of about 100 and 600 years, respectively. If more detailed studies were to confirm these figures, it would mean that the Medium-Head Project would soon become a run-of-the-

river plant. However it will be noted from Section 9 that, even immediately after construction, the Medium-Head Project would have a firm outflow of only 4,300 cfs, compared with an annual mean monthly flow of 17,000 cfs, indicating the Medium-Head Project would not regulate the river to any significant degree. (In the average flow year the spillway would be operating for all months except December, January, February and March and in these months the average inflow is not much less than the firm outflow of 4,300 cfs). Consequently, siltation would have little detrimental effect except on the intake structure, which could well begin to be affected some 40 years after the start of operations. This would require further investigation.

It would appear that the High-Head Project would be little affected by siltation. Even the low level intake structure would not likely be affected owing to its great distance from the upstream end of the reservoir, and since no major tributaries enter the Athabasca River within the reservoir area.

12.11 Ice Conditions

It will be noted from Section 11 that the presence of either postulated dam would have the effect of reducing the volume of ice available to form ice jams in the vicinity of Fort McMurray during the break-up period. In the case of the High-Head Project the Athabasca River would be free of ice from the dam to below the confluence with the Clearwater River, thus preventing the type of flooding which occurred at Fort McMurray in April 1974. This would not be the case with the Medium-Head Project.

The presence of the dam, could cause some minor flooding at Fort McMurray during abnormally warm periods between January and

March with the Medium-Head Dam, but only in February with the High-Head Dam.

12.12 Site Access

Main access to the site would probably be from Fort McMurray, or from a point further south off Highway 63. In any event, some 25 miles of access road would have to be constructed. Much of this would have to be over muskeg resulting in moderately difficult construction.

12.13 Costs and Schedules

The estimated capital costs, as given in Section 8, are about $399 million and $1,1 billion for the Medium-Head and High-Head Projects, respectively. These extremely high costs are, of course, directly related to the problems discussed above.

It will be noted that a significant proportion of the capital cost for each postulated project is the Interest During Construction. This amounts to the impressive sums of about $85 million and $328 million for the Medium-Head and High-Head Projects, respectively. These figures, of course, are affected not only by the rate of interest which must be paid, but also by the total construction time and the cash flow during construction. The rate of interest assumed was 9 per cent per annum, a figure which appears to be appropriate by current standards, but which is greatly dependent upon economic conditions and outlooks. The total construction time and cash flow were estimated on the basis of rough construction schedules, but they would have to be investigated in much greater depth if the postulated projects were to be considered further.

It might be pointed out that one of the factors which would influence the construction time is the relatively short above-ground construction period, particularly for fill placement. This would probably be between 7 and 8 months. Phasing of diversion is another important factor.

It should also be pointed out that considering the complexity of the postulated damsites a long time would be required for investigation and pre-construction engineering. This could amount to up to 3 years in the case of the Medium-Head Project and up to 4 years in the case of the High-Head Project.

Regarding the cost of power, the assumptions made for Annual Charges are given in Section 9. Again the 9 per cent per annum figures prominantly in the calculation.

The estimated costs of energy from either project, but particularly from the Medium-Head Project, are extremely high, being for the Medium-Head and High-Head Projects respectively, 76 mills and 30 mills per kWh for firm energy, and 46 mills and 26 mills per kWh for total energy.

12.15 Downstream Effects

a. General

Because of the problems and costs associated with a development at Crooked Rapids, as discussed above, it was not considered necessary to study the downstream effects at this time. The following, however, are a few general observations on some of the effects which, of course would require to be studied in detail if the project were to

be taken further. Other considerations not mentioned, particularly environmental effects, would also have to be gone into.

b. Flooding

It would appear that the only downstream flooding which occurs at the present time is at Fort McMurray during the ice break-up period. As stated in Section 12.11, this situation would be alleviated by the High-Head Project since open-water would exist from the dam to below Fort McMurray. Furthermore, the amount of ice available to form jams would be greatly reduced with either project.

Consequently, even if some minor flooding were to occur during abnormally warm periods in January to March, which as stated in Section 12.11 is possible, the net effect of either dam would be beneficial from the flooding point of view.

c. Navigation

The effect of either project on navigation on the Athabasca River downstream of Fort McMurray would probably be beneficial in that sand bars would tend not to form and navigation channels would become more incised. Furthermore, in the case of the High-Head Dam, there would be a higher guaranteed flow towards the end of the navigation season.

d. Water Supply and Pollution Control

If anything, the presence of either dam would have a

beneficial effect on the water supply and pollution control situation in that silting of the water intakes would tend to be prevented and that the guaranteed flow would be increased at certain times of the year.

e. Degradation

As stated in Section 12.3d above, it is unlikely the presence of either dam would cause degradation to the extent that would increase slide problems downstream. However, long term degradation at bridge foundations would have to be taken into consideration. It is believed that the water intake and the bridge foundations at Fort McMurray are both on rock so that degradation would not be a problem there.

f. Delta at Lake Athabasca

The effect of the great decrease in silt content of the river could have significant environmental and physical effects at the Delta. (Most of the sediment reaching Lake Athabasca comes from the Athabasca River above Crooked Rapids).

As an example of a possible environmental effect, the silt which is at present reaching the Delta may contain nutrients which, after construction of the dam, would no longer be available. A physical effect, for example, could be an initial sudden build-up of the Delta by material eroded from the riverbed downstream of the dam, followed by a stabilizing of the channels at the Delta.

12.16 Upstream Effects

As in the case of the Downstream Effects, and for the same reasons, the effects of a dam on the area upstream were not studied at this time.

The effects on land use and recreation in the reservoir area were mentioned briefly in Section 12.3c.

It was assumed that clearing would be carried out within the reservoir drawdown zone.

13. CONCLUSIONS AND RECOMMENDATIONS

13.1 There are two locations near Crooked Rapids which, on the basis of topography, are most favourable for consideration as damsites. One of the locations, near the downstream end of the Crooked Rapids meander loop, is topographically suitable for a project incorporating a Medium-Head Dam, while the other, some 5 miles upstream from the first, is topographically suitable for a project incorporating a High-Head Dam. Projects incorporating a 255-ft high dam and a 540-ft high dam were studied at these locations.

13.2 Abutment foundation conditions at both postulated sites are extremely poor. In fact, Dr. Arthur Casagrande, possibly the world's foremost authority on the geotechnical aspects of dam design, who was consulted during the study, has stated:

"The abutment materials (at Crooked Rapids) are much worse and pose more difficult problems than have ever been encountered for a high dam. Whatever design is developed to cope with these problems it has to be very conservative because there are certainly complications that we cannot foresee. Also, I consider it essential that the design must include the possibility of emptying the reservoir to carry out modifications to the seepage control measures".

In his statement, Dr. Casagrande went on to describe certain investigations which he considered to be an essential part of any detailed feasibility study.

13.3 Such abutment conditions rule out the construction of a

concrete dam of any type. Any dam constructed in the vicinity of Crooked Rapids would thus have to be of earth, or a combination of earth and rock. However, there are no sizeable deposits of good earth borrow materials near Crooked Rapids. Hence, a dam would have to be constructed largely from locally excavated rocks which would include the oil-rich Athabasca Tar Sands, sandy materials from required project excavations, and quarried limestone. The limestone, although by far the most competent rock in the area is fragmental and shaley, factors which detract from its suitability as fill material.

13.4 It is clear from the above, and from the discussion in Section 12, that there are severe technical problems associated with any postulated development at Crooked Rapids, and that several of the features shown on the project layouts are unprecedented. These include the 50-ft excavated diameter diversion/power tunnels through the Tar Sands, the use of the Tar Sands as an impervious core of the dam, and the extremely deep and extensive seepage cut-off wall for the High-Head Project.

13.5 The known and potential problems associated with both postulated developments are of such a magnitude that a much more detailed investigation, especially for the High-Head Project, would be required before it could definitely be determined if it is feasible to develop either of the sites. Such investigation would include exploratory drilling, exploratory adits, laboratory testing and further design studies.

13.6 The above notwithstanding, it is believed with a reasonable degree of confidence that the problems associated with the Medium-Head Project could be overcome and the project could be

constructed within the capital cost estimated. In fact, it is possible that the estimated capital cost could prove to be somewhat conservative if the oil-rich Tar Sands and the underlying limestone turned out to be more competent than can be assumed at this time.

13.7 However, the same cannot be said for the High-Head Project. Many of the problems are considerably more severe for the High-Head Project than for the Medium-Head Project. For example, construction of an effective seepage cut-off for the High-Head Project would be an extraordinarily difficult task which, upon detailed investigation, might prove not to be technically feasible at reasonable cost, whereas the required cut-off wall for the Medium-Head Project would be well within precedence. Therefore, because of the more severe problems and the resulting greater uncertainties, the High-Head Project layout must at this stage of the investigation be considered "conceptual" only.

13.8 The total capital cost of the postulated projects in terms of January, 1975 dollars is estimated at about 400 million for the Medium-Head Project and 1.1 billion for the High-Head Project.

13.9 For the Medium-Head Project, the estimated cost per installed kW is $3,320, and the estimated firm and total energy costs, not including transmission, are 76 mills and 46 mills per kWh, respectively. These costs are extremely high, and would remain so even if the estimated capital cost of the project proved in fact to be somewhat conservative as discussed above.

13.10 For the High-Head Project the estimated cost per installed kW is $2,000, and the estimated firm and total energy costs, not including transmission, are 30 mills and 24 mills per kWh, respectively. These costs are much less than for the Medium-Head Project but are still very high when compared with current costs.

13.11 Although limited studies indicate that there would be some non power-related benefits from construction of a dam at Crooked Rapids, for example the potential for flooding at Fort McMurray would be decreased and navigation channels would tend to be improved, these do not appear to be substantial enough to be significant factors in justifying the very high capital expenditures which would be involved at Crooked Rapids. Such expenditures would have to be justified mainly on the basis of benefits derived from the energy output. Although more would have to be known regarding the specific uses for the power, it is most unlikely, considering the energy costs given above, that a development at Crooked Rapids, without the benefit of upstream storage, could be justified at this time.

13.12 It is recommended, therefore, that before detailed studies are carried out at Crooked Rapids, an overall appraisal of the Athabasca River be made in effort to:

a. locate alternate sites to Crooked Rapids, where geological conditions and the availability of borrow materials are more favourable, and/or

b. to locate sites which could provide a development at Crooked Rapids with upstream storage, and thus make development there more economical.

Such an appraisal would comprise an airphoto study of the entire river, geological mapping at selected locations, and preliminary feasibility studies at probably two or three potential damsites.

Initially these feasibility studies could be more preliminary than the Crooked Rapids study. The hydrological investigations carried out for the Crooked Rapids study would provide a good deal of the hydrologic information required for the overall appraisal of the river. However, if an overall appraisal is made it would be advisable to obtain better topography along the entire river, so that reservoir storage volumes could be more accurately obtained. If this is done the reservoir storage curves for Crooked Rapids should be re-estimated, and if necessary the power and energy studies refined.

14. ACKNOWLEDGEMENTS

The Consultants wish to acknowledge the assistance given by Calgary Power Ltd. in making available to them various reports on studies the company had made of the power potential of the Athabasca River, and for supplying them with maps of the reservoir area from which, as noted on Figs. 6 and 9, the reservoir storage curves were obtained. These maps were prepared in 1953 by Calgary Power Ltd. working in conjunction with the Alberta Power Commission, and form part of the study referred to in Section 1.3.

They should also like to acknowledge the assistance given by the Alberta Research Council and the Alberta Department of Highways who supplied information on the soil conditions in the Fort McMurray area.

PHOTOS

General view of Damsite Study Area.
The Athabasca River flows towards right.
Note ancient meander in front of present
Crooked Rapids meander loop.

PHOTO 1

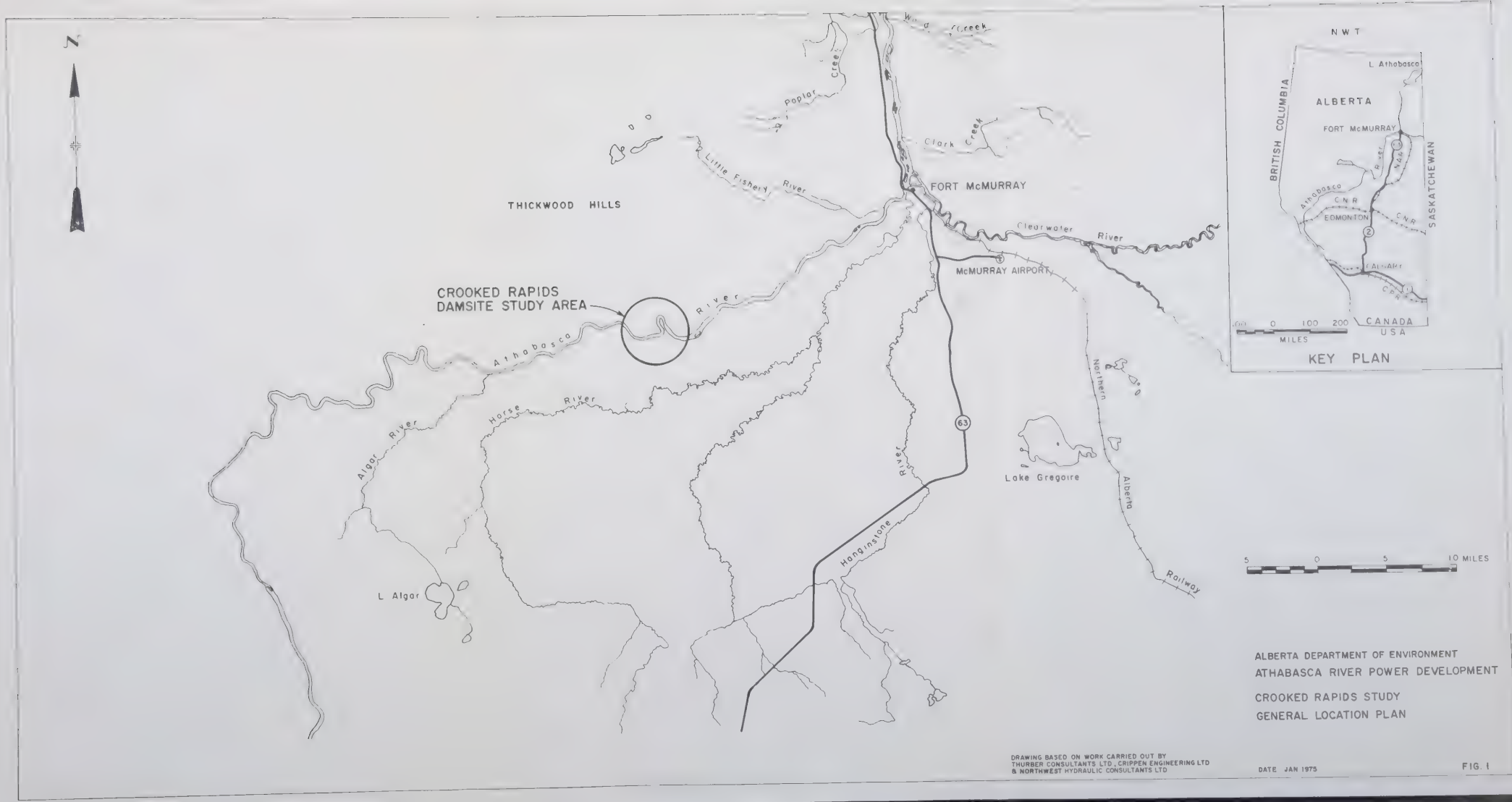
View of Medium-Head Site.
The Athabasca River flows towards left.

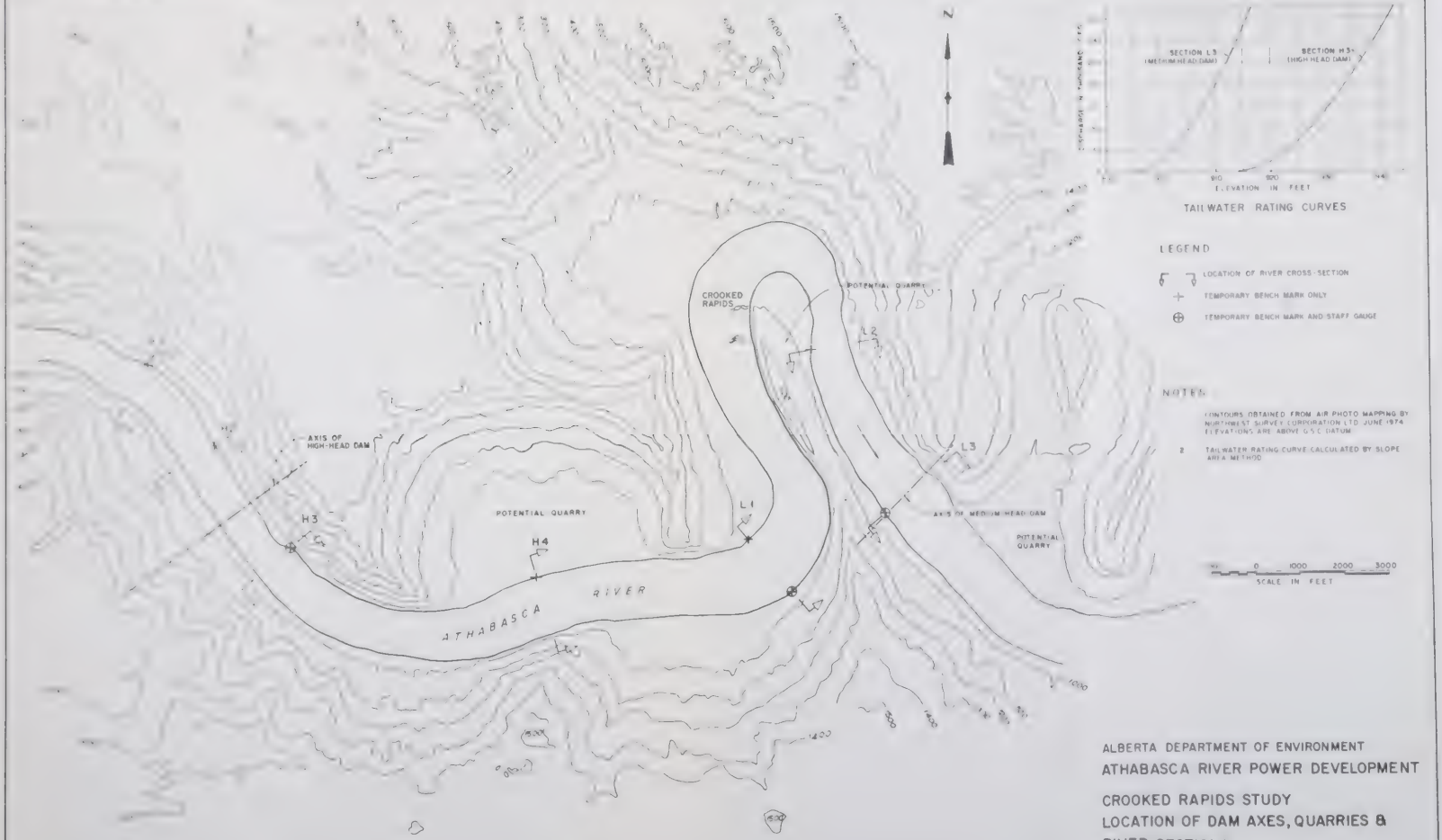
PHOTO 2

View of High-Head Site.
The Athabasca River flows towards right.

PHOTO 3

FIGURES


N
Poplar Creek
Clark Creek
Little Fishery River
THICKWOOD HILLS
FORT McMURRAY
Clearwater River
McMURRAY AIRPORT
CROOKED RAPIDS
DAMSITE STUDY AREA
Athabasca River
Algar River
Horse River
L Algar
63
Hangingstone River
Lake Gregoire
Northern Alberta Railway
NWT
L Athabasca
ALBERTA
BRITISH COLUMBIA
SASKATCHEWAN
FORT McMURRAY
Athabasca River
C.N.R.
EDMONTON
CALGARY
CANADA
USA
MILES
KEY PLAN
5 0 5 10 MILES
ALBERTA DEPARTMENT OF ENVIRONMENT
ATHABASCA RIVER POWER DEVELOPMENT
CROOKED RAPIDS STUDY
GENERAL LOCATION PLAN
DRAWING BASED ON WORK CARRIED OUT BY
THURBER CONSULTANTS LTD., CRIPPEN ENGINEERING LTD
& NORTHWEST HYDRAULIC CONSULTANTS LTD
DATE JAN 1975
FIG. 1



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ATHABASCA RIVER POWER DEVELOPMENT

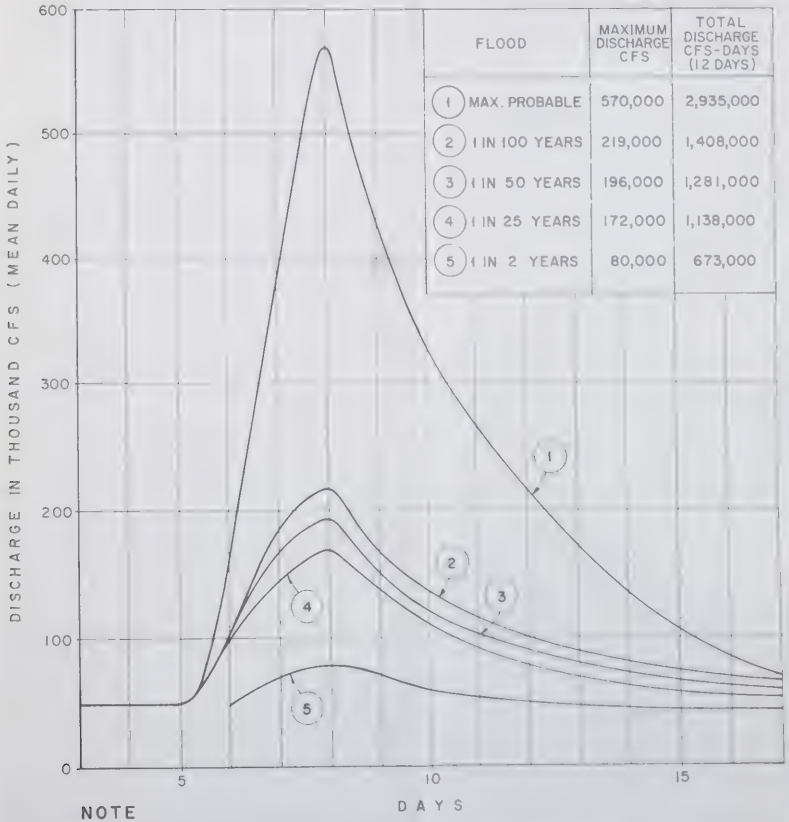
CROOKED RAPIDS STUDY

LOCATION OF DAM AXES, QUARRIES & RIVER SECTIONS

DRAWING BASED ON WORK CARRIED OUT BY CRIPPEN ENGINEERING LTD & NORTHWEST HYDRAULIC CONSULTANTS LTD

DATE JAN 1975

FIG 2



FLOOD	MAXIMUM DISCHARGE CFS	TOTAL DISCHARGE CFS-DAYS (12 DAYS)
(1) MAX. PROBABLE	570,000	2,935,000
(2) 1 IN 100 YEARS	219,000	1,408,000
(3) 1 IN 50 YEARS	196,000	1,281,000
(4) 1 IN 25 YEARS	172,000	1,138,000
(5) 1 IN 2 YEARS	80,000	673,000

NOTE

DAY ZERO IS START OF DIRECT RUNOFF IN HEADWATERS

ALBERTA DEPARTMENT OF ENVIRONMENT

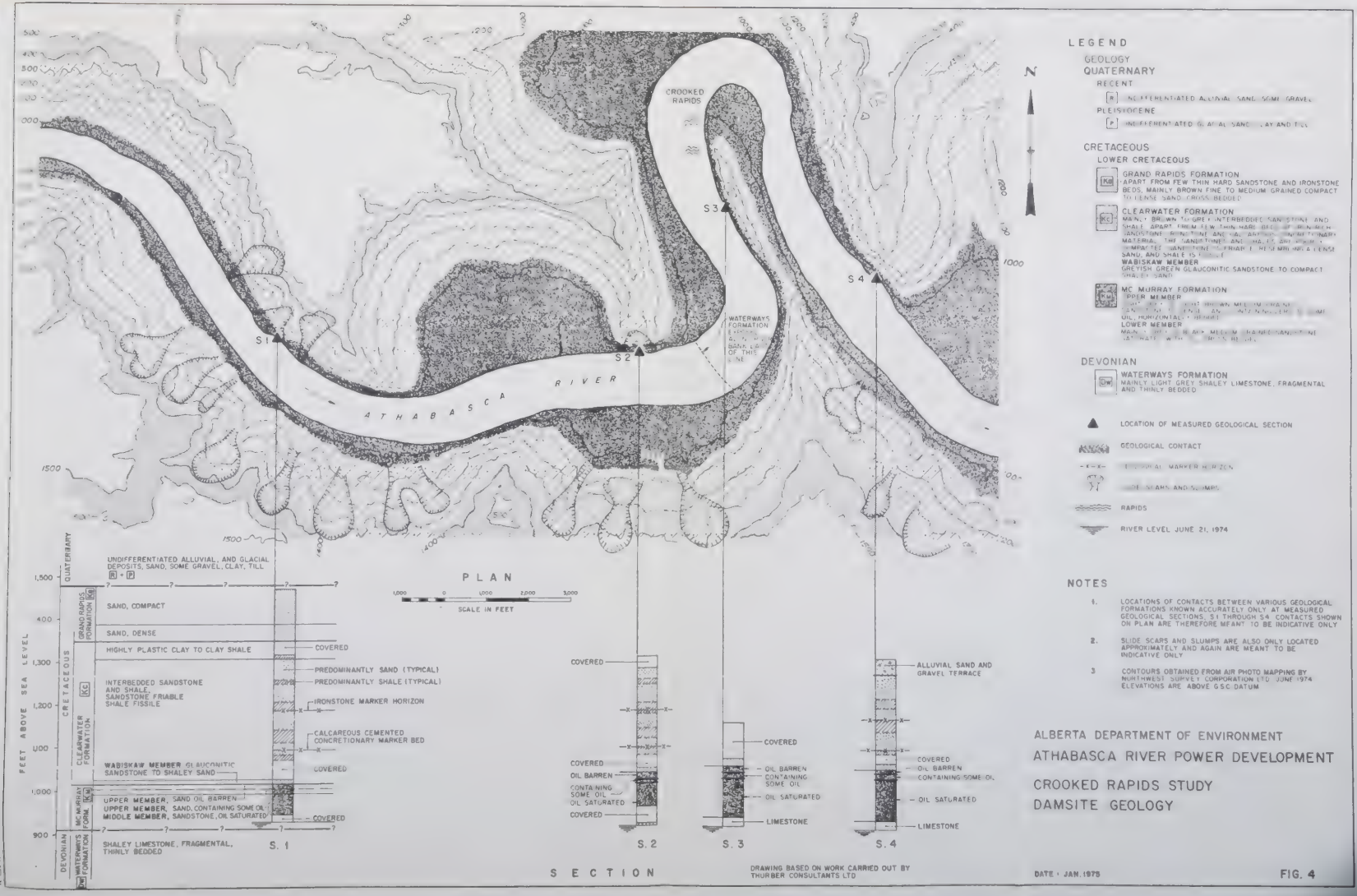
ATHABASCA RIVER POWER DEVELOPMENT

CROOKED RAPIDS STUDY

FLOOD HYDROGRAPHS

DRAWING BASED ON WORK CARRIED OUT BY NORTHWEST HYDRAULIC CONSULTANTS LTD.

FIG. 3



LEGEND

GEOLOGY

QUATERNARY

RECENT

(R) UNDIFFERENTIATED ALLUVIAL SAND, SOME GRAVEL

PLEISTOCENE

(P) UNDIFFERENTIATED GLACIAL SAND, CLAY AND TILL

CRETACEOUS

LOWER CRETACEOUS

(Kg) GRAND RAPIDS FORMATION
APART FROM FEW THIN HARD SANDSTONE AND IRONSTONE BEDS, MAINLY BROWN FINE TO MEDIUM GRAINED COMPACT TO LENSE SAND, CROSS BEDDED

(Kc) CLEARWATER FORMATION
MAINLY BROWN TO GREY INTERBEDDED SANDSTONE AND SHALE APART FROM FEW THIN HARD BEDS OF BROWN IRONSTONE, SIDERITE AND CALCAREOUS CONCRETIONARY MATERIAL. THE SANDSTONE AND SHALE ARE PRIMARILY COMPACT, SANDSTONE FRIABLE, FISSILE AND A LENSE SAND, AND SHALE IS [illegible]
WABISKAW MEMBER
GREYISH GREEN GLAUCONITIC SANDSTONE TO COMPACT SHALEY SAND

(Km) MC MURRAY FORMATION
UPPER MEMBER
[illegible]
LOWER MEMBER
MAIN [illegible] MEDIUM GRAINED SANDSTONE [illegible]

DEVONIAN

(Dw) WATERWAYS FORMATION
MAINLY LIGHT GREY SHALEY LIMESTONE, FRAGMENTAL AND THINLY BEDDED

▲ LOCATION OF MEASURED GEOLOGICAL SECTION

GEOLOGICAL CONTACT

[illegible] GEOLOGICAL MARKER HORIZON

[illegible] SLIDE SCARPS AND SLUMPS

RAPIDS

RIVER LEVEL JUNE 21, 1974

NOTES

1. LOCATIONS OF CONTACTS BETWEEN VARIOUS GEOLOGICAL FORMATIONS KNOWN ACCURATELY ONLY AT MEASURED GEOLOGICAL SECTIONS, S.1 THROUGH S.4. CONTACTS SHOWN ON PLAN ARE THEREFORE MEANT TO BE INDICATIVE ONLY
2. SLIDE SCARPS AND SLUMPS ARE ALSO ONLY LOCATED APPROXIMATELY AND AGAIN ARE MEANT TO BE INDICATIVE ONLY
3. CONTOURS OBTAINED FROM AIR PHOTO MAPPING BY NORTHWEST SURVEY CORPORATION LTD. JUNE 1974. ELEVATIONS ARE ABOVE G.S.C. DATUM

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ATHABASCA RIVER POWER DEVELOPMENT

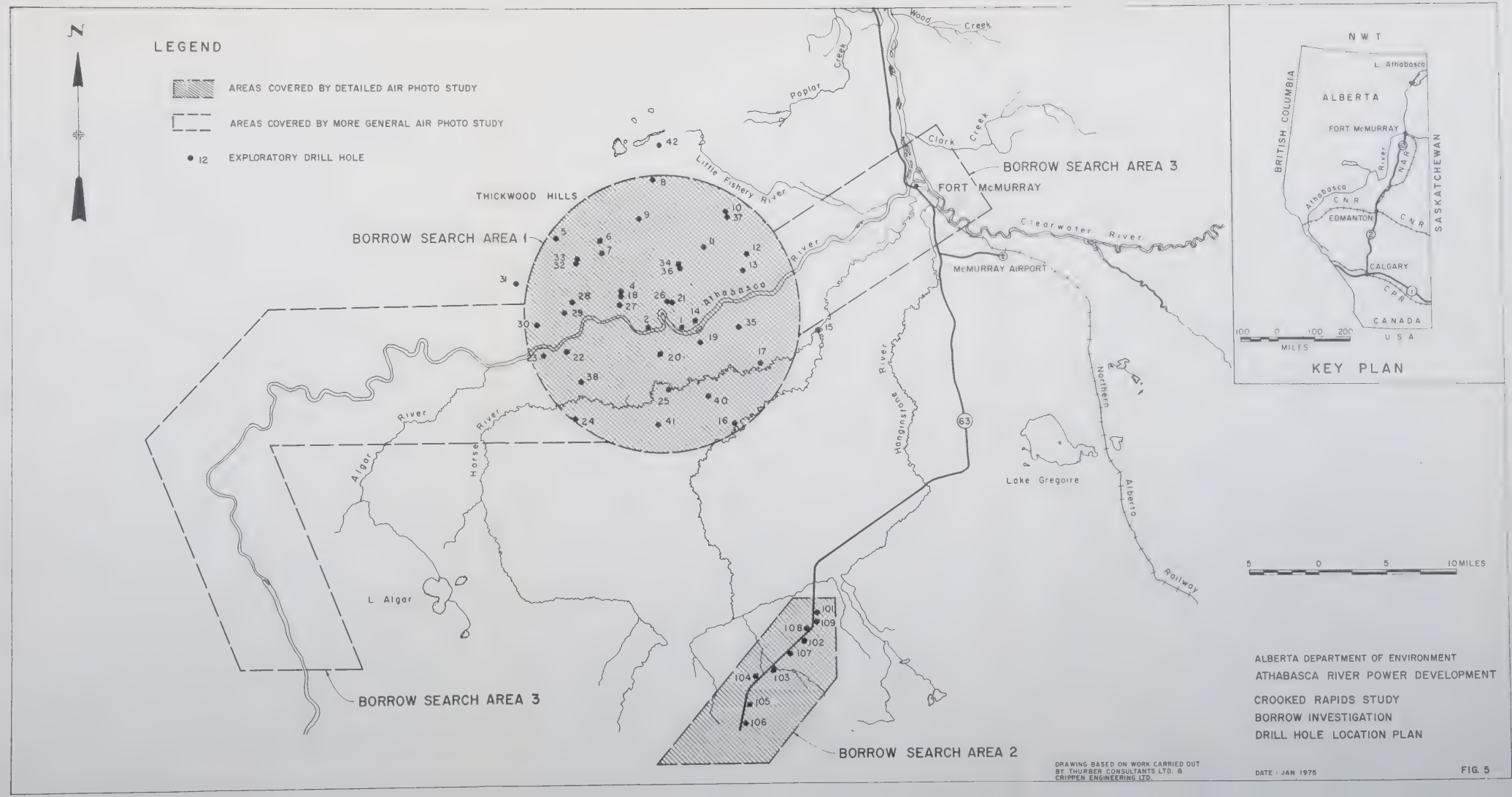
CROOKED RAPIDS STUDY

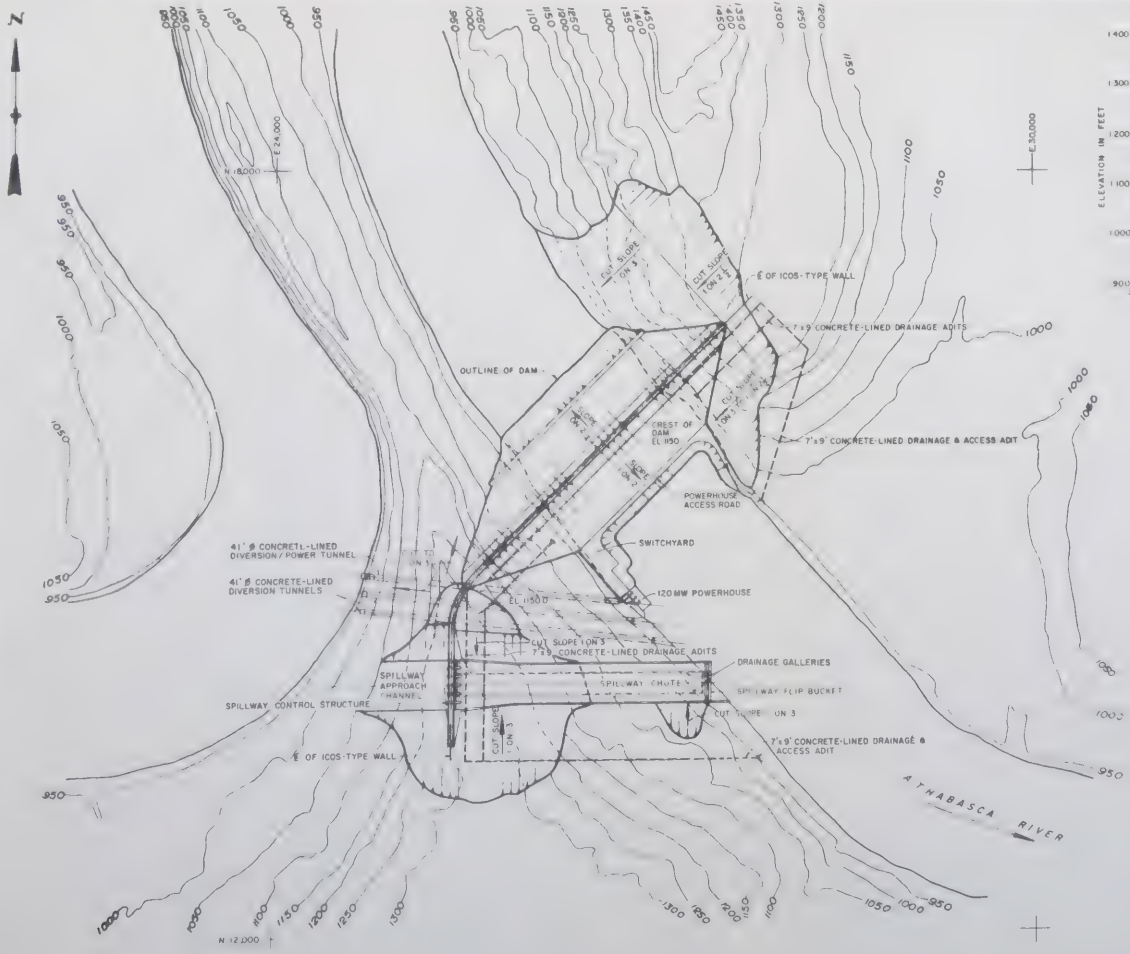
DAMSITE GEOLOGY

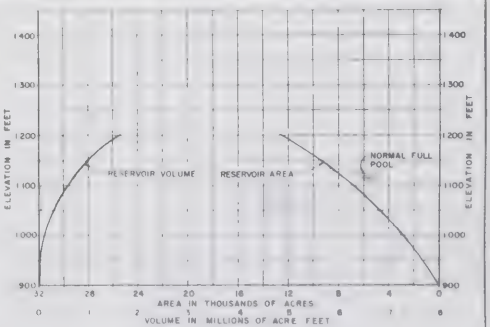
DRAWING BASED ON WORK CARRIED OUT BY THURBER CONSULTANTS LTD

DATE : JAN. 1975

FIG. 4


N
LEGEND
AREAS COVERED BY DETAILED AIR PHOTO STUDY
AREAS COVERED BY MORE GENERAL AIR PHOTO STUDY
• 12 EXPLORATORY DRILL HOLE
THICKWOOD HILLS
BORROW SEARCH AREA 1
BORROW SEARCH AREA 3
BORROW SEARCH AREA 3
BORROW SEARCH AREA 2
FORT McMURRAY
McMURRAY AIRPORT
Clearwater River
Athabasca River
Little Fishery River
Wood Creek
Poplar Creek
Clark Creek
Algar River
Horse River
Hangingstone River
Lake Gregoire
L Algor
Northern Alberta Railway
63
5 0 5 10 MILES
NWT
L Athabasca
ALBERTA
FORT McMURRAY
BRITISH COLUMBIA
SASKATCHEWAN
Athabasca River
N A R
C N R
EDMANTON
CALGARY
C P R
CANADA
U S A
100 0 100 200
MILES
KEY PLAN
ALBERTA DEPARTMENT OF ENVIRONMENT
ATHABASCA RIVER POWER DEVELOPMENT
CROOKED RAPIDS STUDY
BORROW INVESTIGATION
DRILL HOLE LOCATION PLAN
DRAWING BASED ON WORK CARRIED OUT BY THURBER CONSULTANTS LTD. & CRIPPEN ENGINEERING LTD.
DATE : JAN. 1975
FIG. 5





RESERVOIR STORAGE CURVES

NOTES

1. FOR SECTIONS OF DAM SEE FIGURE 7
2. FOR SECTIONS ALONG SPILLWAY AND TUNNELS, AND THROUGH POWERHOUSE SEE FIGURE 8
3. SLOPES ABOVE UPSTREAM PORTALS OF TUNNELS AND ABOVE UPSTREAM SLOPE OF DAM ON RIGHT BANK TO BE CUT TO 1 ON 3. FOR CLARITY CUT LINES NOT SHOWN ON PLAN. SEE SECTIONS FIGURES 7 & 8
4. FOR SLOPE OF FILL OVER CUT AND COVER SECTIONS OF TUNNELS SEE FIGURE 8
5. CONTOURS OBTAINED FROM AIR PHOTO MAPPING BY NORTHWEST SURVEY CORPORATION LIMITED, JUNE 1974. ELEVATIONS ARE ABOVE GSC DATUM
6. RESERVOIR VOLUME CURVES OBTAINED FROM ROUGH CONTOUR MAPS SUPPLIED BY CALGARY POWER LTD
7. SPILLWAY APPROACH CHANNEL & CHUTE TO BE LINED
8. SITE ROADS NOT SHOWN

400 0 400 800 1200
SCALE IN FEET

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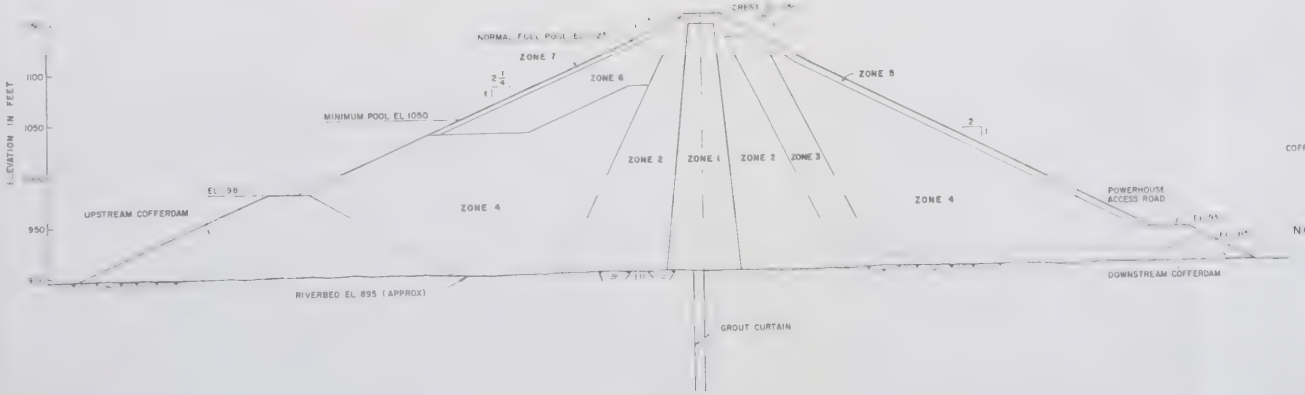
ATHABASCA RIVER POWER DEVELOPMENT

CROOKED RAPIDS STUDY
MEDIUM-HEAD PROJECT
GENERAL ARRANGEMENT

DRAWING BASED ON WORK CARRIED OUT BY CRIPPEN ENGINEERING LTD

DATE · JAN. 1975

FIG. 6



DAM CROSS-SECTION AT MAXIMUM HEIGHT

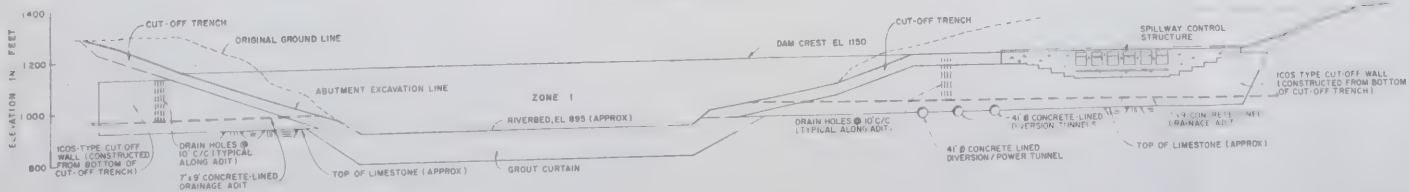
50 0 50 100 150
SCALE IN FEET

LEGEND

ZONE	MATERIAL
1	RICH TAR SANDS
2	QUARRIED LIMESTONE FINES, OR SANDY MATERIAL FROM REQUIRED EXCAVATIONS
3	CLEAN FILTER AND DRAIN GRAVEL
4	QUARRY-RUN LIMESTONE (COARSER MATERIAL TOWARDS OUTSIDE SLOPES)
5	CLEAN QUARRIED LARGE-SIZED LIMESTONE
6	CLEAN QUARRIED MEDIUM-SIZED LIMESTONE
7	RIPRAP
COFFERDAMS	MAINLY ROCK-ZONING TO BE DECIDED ON BASIS OF DIVERSION SCHEME

NOTES

1. FOR LOCATION OF SECTIONS SEE FIGURE 6
2. FOR SECTIONS ALONG TUNNELS AND SPILLWAY SEE FIGURE 8
3. SLOPES AND DIMENSIONS OF FILL ZONES MAY VARY SOMEWHAT FROM THOSE SHOWN. QUANTITIES BASED ON SLOPES & DIMENSIONS SHOWN
4. ZONE 3 TO BE CARRIED UP ABUTMENT UNDER DOWNSTREAM SHELL
5. ALL FILL MATERIALS, EXCEPT RIPRAP & OTHER LARGE ROCK NEAR EXTERIOR SLOPES, AND PARTS OF COFFERDAMS, TO BE PLACED IN LIFTS AND COMPACTED
6. DRAINAGE ADITS AND DRAIN HOLES PROJECTED ON TO SECTION
7. CUT-OFF TRENCH TO BE BACKFILLED WITH IMPERVIOUS MATERIAL


ELEVATION IN FEET
1400
1200
1000
800
CUT-OFF TRENCH
ORIGINAL GROUND LINE
ABUTMENT EXCAVATION LINE
ZONE 1
DAM CREST EL 1150
CUT-OFF TRENCH
SPILLWAY CONTROL STRUCTURE
ICOS TYPE CUT-OFF WALL (CONSTRUCTED FROM BOTTOM OF CUT-OFF TRENCH)
RIVERBED, EL 895 (APPROX)
ICOS-TYPE CUT-OFF WALL (CONSTRUCTED FROM BOTTOM OF CUT-OFF TRENCH)
DRAIN HOLES @ 10' C/C (TYPICAL ALONG ADIT)
7'x9' CONCRETE-LINED DRAINAGE ADIT
TOP OF LIMESTONE (APPROX)
GROUT CURTAIN
DRAIN HOLES @ 10' C/C (TYPICAL ALONG ADIT)
41' Ø CONCRETE LINED DIVERSION TUNNEL
41' Ø CONCRETE LINED DIVERSION / POWER TUNNEL
TOP OF LIMESTONE (APPROX)

SECTION ALONG DAM AXIS

200 0 200 400 600
SCALE IN FEET

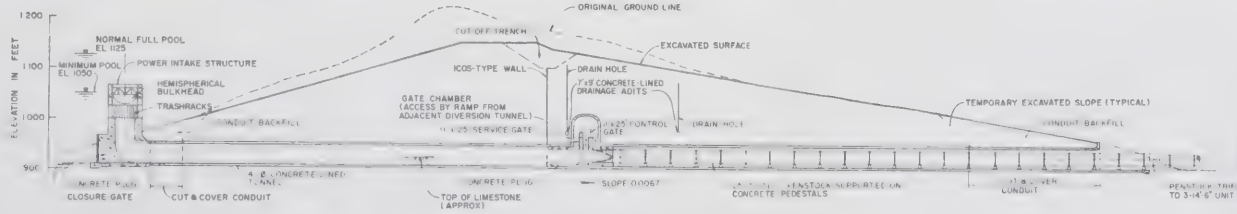
ALBERTA DEPARTMENT OF ENVIRONMENT
ATHABASCA RIVER POWER DEVELOPMENT

CROOKED RAPIDS STUDY
MEDIUM-HEAD PROJECT
DAM SECTIONS

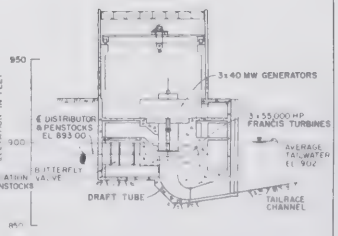
DRAWING BASED ON WORK CARRIED OUT BY CRIPPEN ENGINEERING LTD.

DATE JAN. 1975

FIG. 7

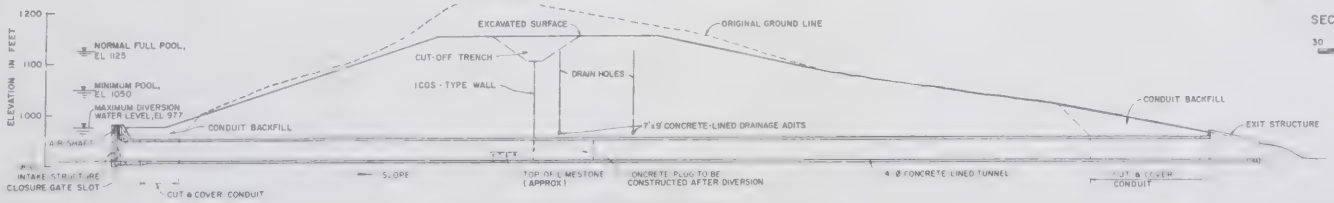


SECTION THROUGH DIVERSION / POWER TUNNEL



SECTION THROUGH POWERHOUSE

30 0 30 60 90
SCALE IN FEET



SECTION THROUGH DIVERSION TUNNELS
(TYPICAL FOR TWO TUNNELS)

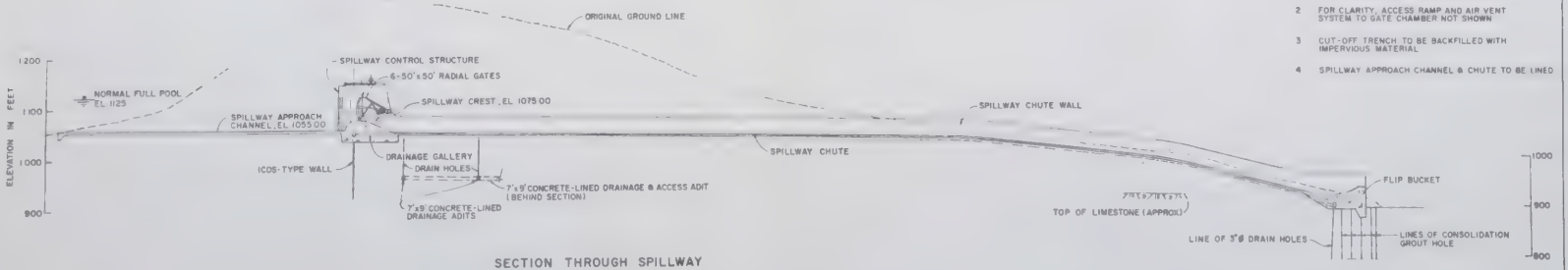
NOTES

1 FOR LOCATION OF SECTIONS SEE FIGURE 6

2 FOR CLARITY, ACCESS RAMP AND AIR VENT SYSTEM TO GATE CHAMBER NOT SHOWN

3 CUT-OFF TRENCH TO BE BACKFILLED WITH IMPERVIOUS MATERIAL

4 SPILLWAY APPROACH CHANNEL & CHUTE TO BE LINED



SECTION THROUGH SPILLWAY

100 0 100 200 300 400
SCALE IN FEET

ALBERTA DEPARTMENT OF ENVIRONMENT

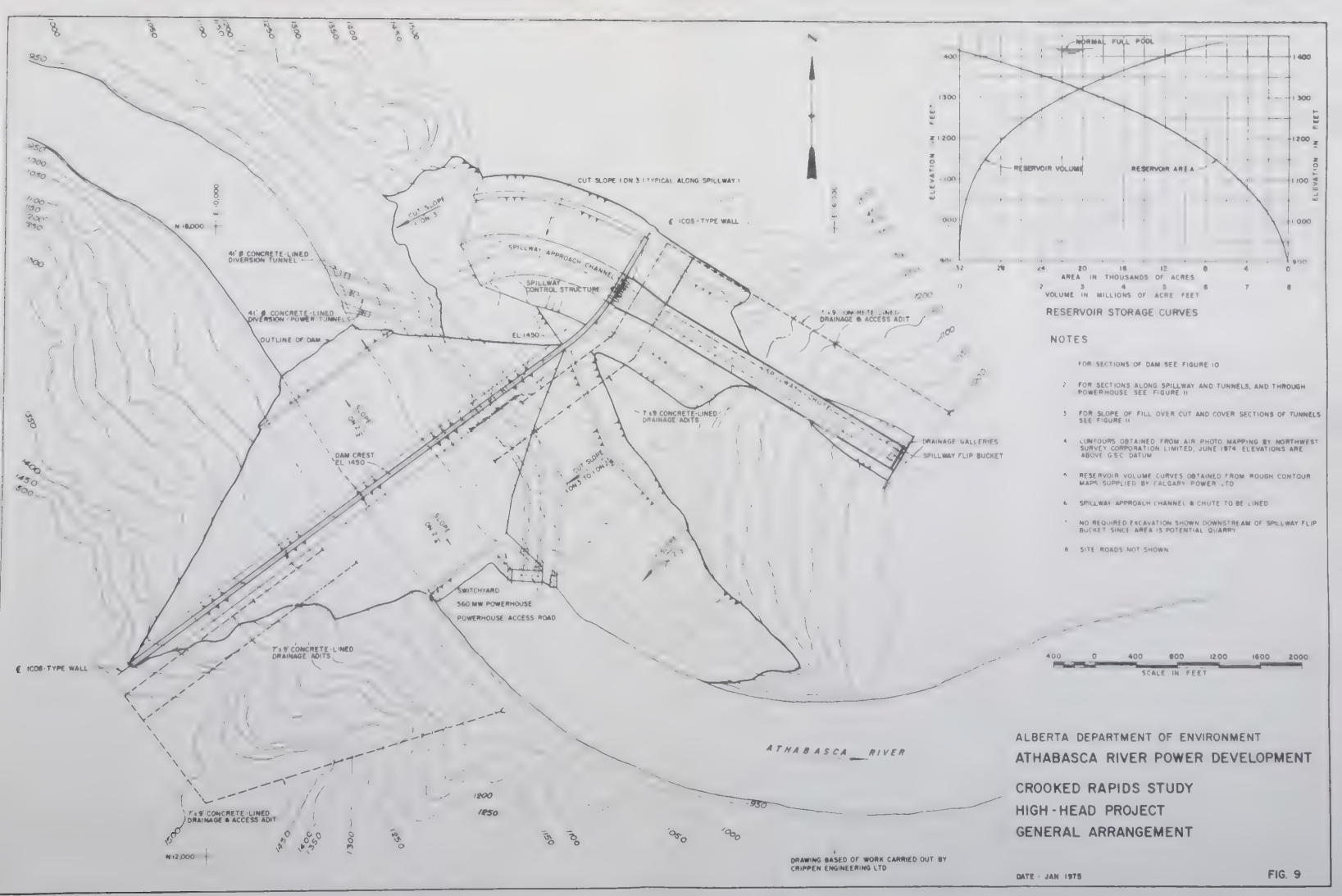
ATHABASCA RIVER POWER DEVELOPMENT

CROOKED RAPIDS STUDY
MEDIUM-HEAD PROJECT
SECTIONS THROUGH TUNNELS & SPILLWAY

DRAWING BASED ON WORK CARRIED OUT BY CRIPPEN ENGINEERING LTD.

DATE : JAN 1975

FIG. 8



RESERVOIR STORAGE CURVES

NOTES

1. FOR SECTIONS OF DAM SEE FIGURE 10
2. FOR SECTIONS ALONG SPILLWAY AND TUNNELS, AND THROUGH POWERHOUSE SEE FIGURE 11
3. FOR SLOPE OF FILL OVER CUT AND COVER SECTIONS OF TUNNELS SEE FIGURE 11
4. CONTOURS OBTAINED FROM AIR PHOTO MAPPING BY NORTHWEST SURVEY CORPORATION LIMITED, JUNE 1974. ELEVATIONS ARE ABOVE G.S.C. DATUM
5. RESERVOIR VOLUME CURVES OBTAINED FROM ROUGH CONTOUR MAPS SUPPLIED BY CALGARY POWER LTD
6. SPILLWAY APPROACH CHANNEL & CHUTE TO BE LINED
7. NO REQUIRED EXCAVATION SHOWN DOWNSTREAM OF SPILLWAY FLIP BUCKET SINCE AREA IS POTENTIAL QUARRY
8. SITE ROADS NOT SHOWN

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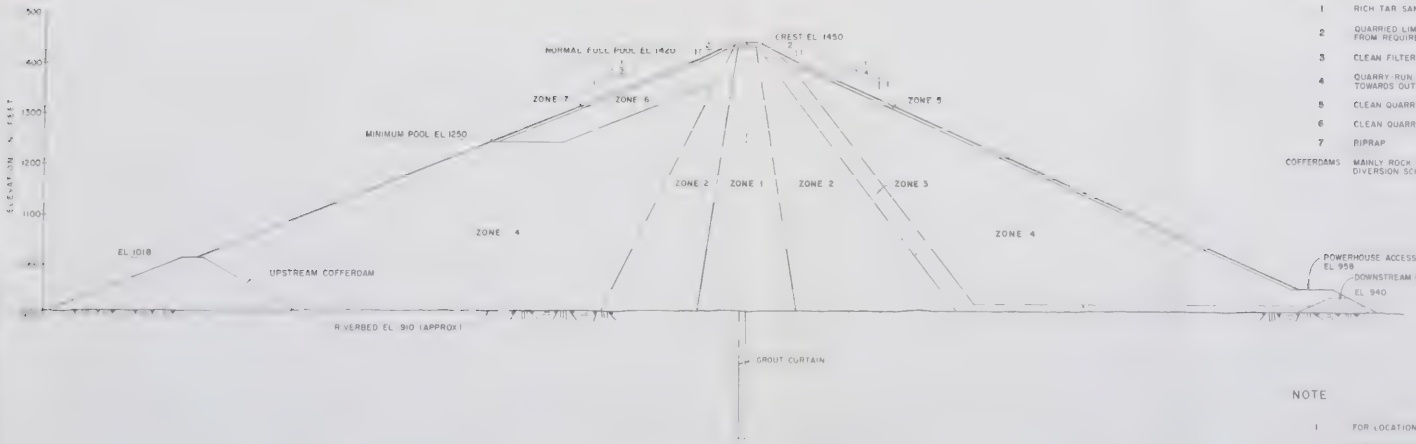
ALBERTA DEPARTMENT OF ENVIRONMENT

ATHABASCA RIVER POWER DEVELOPMENT

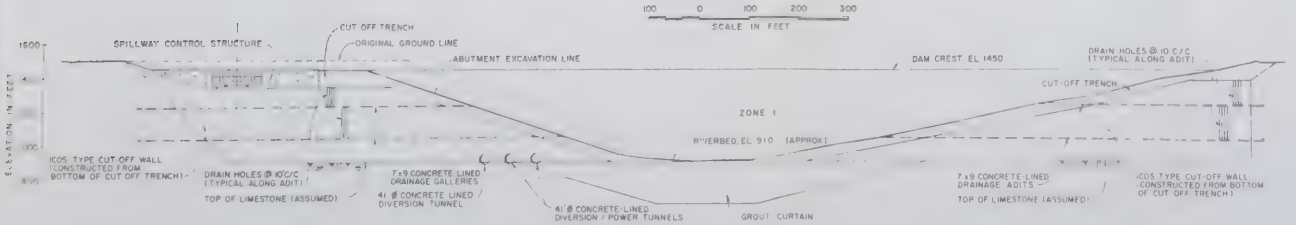
CROOKED RAPIDS STUDY
HIGH-HEAD PROJECT
GENERAL ARRANGEMENT

DATE · JAN 1975

FIG. 9



DAM CROSS-SECTION AT MAXIMUM HEIGHT


100 0 100 200 300
SCALE IN FEET
SPILLWAY CONTROL STRUCTURE
CUT OFF TRENCH
ORIGINAL GROUND LINE
ABUTMENT EXCAVATION LINE
DAM CREST EL 1450
DRAIN HOLES @ 10 C/C (TYPICAL ALONG ADIT)
CUT-OFF TRENCH
ZONE 1
RIVERBED EL 910 (APPROX)
ELEVATION IN FEET
ICOS TYPE CUT-OFF WALL (CONSTRUCTED FROM BOTTOM OF CUT OFF TRENCH)
DRAIN HOLES @ 10'C/C (TYPICAL ALONG ADIT)
TOP OF LIMESTONE (ASSUMED)
7'x9' CONCRETE LINED DRAINAGE GALLERIES
41' Ø CONCRETE LINED DIVERSION TUNNEL
41' Ø CONCRETE-LINED DIVERSION / POWER TUNNELS
GROUT CURTAIN
7'x9' CONCRETE-LINED DRAINAGE ADITS
TOP OF LIMESTONE (ASSUMED)
ICOS TYPE CUT-OFF WALL (CONSTRUCTED FROM BOTTOM OF CUT OFF TRENCH)

SECTION ALONG DAM AXIS

LEGEND

ZONE	MATERIAL
1	RICH TAR SANDS
2	QUARRIED LIMESTONE FINES OR SANDY MATERIAL FROM REQUIRED EXCAVATIONS
3	CLEAN FILTER AND DRAIN GRAVEL
4	QUARRY-RUN LIMESTONE (COARSER MATERIAL TOWARDS OUTSIDE SLOPES)
5	CLEAN QUARRIED LARGE SIZED LIMESTONE
6	CLEAN QUARRIED MEDIUM-SIZED LIMESTONE
7	RIPRAP
COFFERDAMS	MAINLY ROCK ZONING TO BE DECIDED ON BASIS OF DIVERSION SCHEME

NOTE

1. FOR LOCATION OF SECTIONS SEE FIGURE 9
2. FOR SECTIONS ALONG TUNNELS AND SPILLWAY SEE FIGURE 11
3. SLOPES AND DIMENSIONS OF FILL ZONES MAY VARY SOMEWHAT FROM THOSE SHOWN. QUANTITIES BASED ON SLOPES & DIMENSIONS SHOWN
4. ZONE 3 TO BE CARRIED UP ABUTMENT UNDER DOWNSTREAM SHELL
5. ALL FILL MATERIALS, EXCEPT RIPRAP & OTHER LARGE ROCK NEAR EXTERIOR SLOPES AND PARTS OF COFFERDAMS, TO BE PLACED IN LIFTS AND COMPACTED
6. EXCAVATION ON RIGHT ABUTMENT OF LIMITED EXTENT NOT SHOWN
7. DRAINAGE ADITS AND DRAIN HOLES, PROJECTED ON TO SECTION
8. CUT-OFF TRENCH TO BE BACKFILLED WITH IMPERVIOUS MATERIAL

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ATHABASCA RIVER POWER DEVELOPMENT

CROOKED RAPIDS STUDY

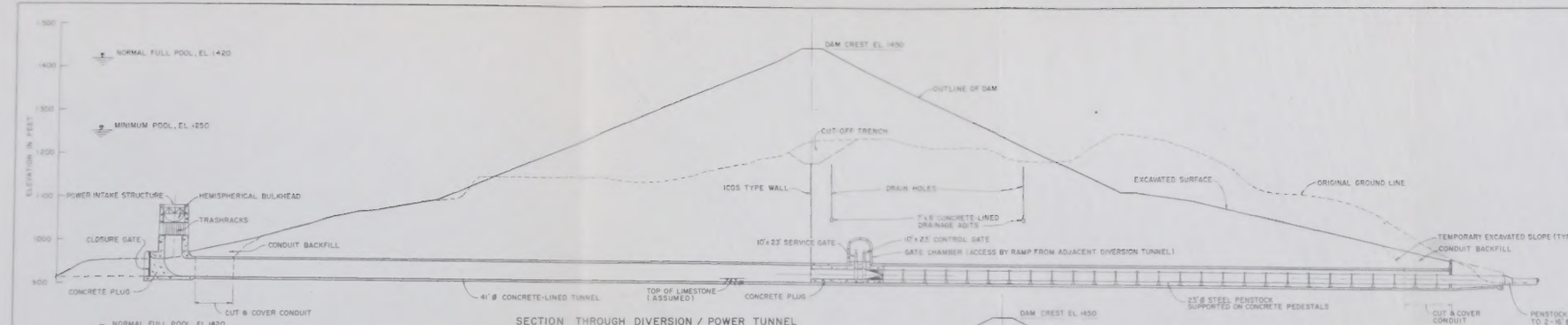
HIGH-HEAD PROJECT

DAM SECTIONS

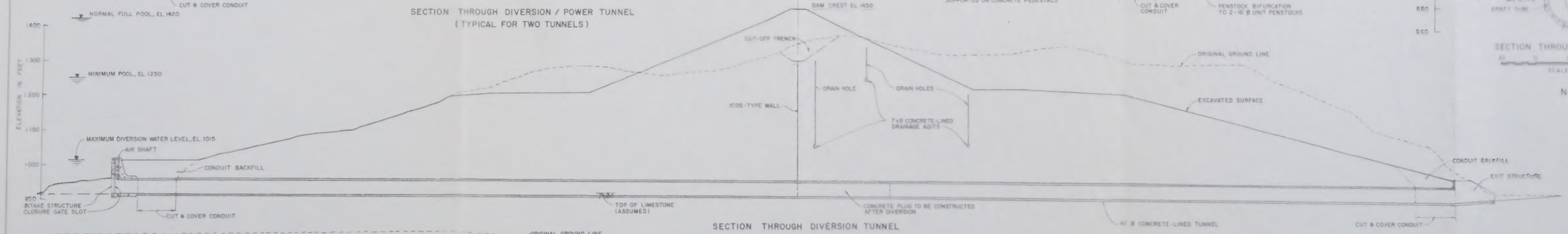
DRAWING BASED ON WORK CARRIED OUT BY CRIPPEN ENGINEERING LTD.

DATE JAN 1975

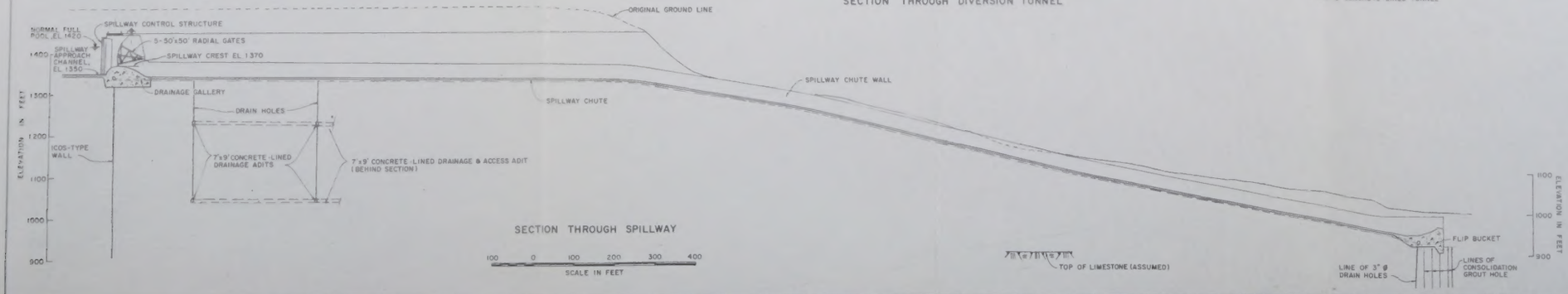
FIG. 10



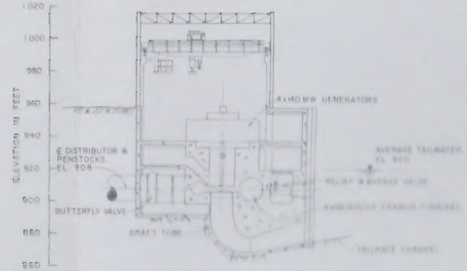
SECTION THROUGH DIVERSION / POWER TUNNEL
(TYPICAL FOR TWO TUNNELS)



SECTION THROUGH DIVERSION TUNNEL



SECTION THROUGH SPILLWAY



SECTION THROUGH POWERHOUSE

NOTES

1. FOR LOCATION OF SECTIONS SEE FIGURE 9
2. FOR CLARITY, ACCESS RAMP AND AIR VENT SYSTEM TO GATE CHAMBER NOT SHOWN
3. CUT-OFF TRENCH TO BE BACKFILLED WITH IMPERVIOUS MATERIAL
4. SPILLWAY APPROACH CHANNEL & CHUTE TO BE LINED

ALBERTA DEPARTMENT OF ENVIRONMENT

ATHABASCA RIVER POWER DEVELOPMENT

CROOKED RAPIDS STUDY
HIGH-HEAD PROJECT
SECTIONS THROUGH TUNNELS & SPILLWAY

DRAWING BASED ON WORK CARRIED OUT BY CRIPPEN ENGINEERING LTD.

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FIG. 11